VOL. 55. MARCH, 1929. No. 3

PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

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ANNUAL MEETING

JANUARY 17, 1929

DIGEST OF TECHNICAL PAPERS AND DISCUSSIONS, AND REPORTS OF COMMITTEES

Every Society meeting furnishes a focus for up-to-date views on diverse engineering problems. The following account of the 1929 Annual Meeting has been prepared in order that the gist of the technical accomplishments may be made promptly available to members. Discussion covering the reports of Society Committees will be welcome for possible future publication in *Proceedings*.

HIGHWAY DIVISION

JANUARY 17, 1929, 10:00 A. M. TO 1:00 P. M.

PROBLEMS OF TRAFFIC REGULATION AND CONTROL

By Leslie G. Holleran,* M. Am. Soc. C. E.

In the unexpected absence of the announced speaker, Mr. Holleran kindly consented to introduce this topic. According to Mr. Holleran:

"Too many highway engineers and traffic officials are still holding to the idea that the beginning and end of traffic control is in the adoption of a sheaf of traffic ordinances, the putting of a squad of traffic policemen on the streets, and the installation of a system of traffic lights. The problem is very much broader than that and is fundamentally a problem of planning and design.

"Some one has said that country is best governed which has the least government, the least legislation, and the least regulation. There is no branch of the highway problem to which this principle more strongly applies than to the problem of traffic control; that is, the efficiency of traffic control ought to be measured by the absence of police and stop lights. If we accept that principle what then?

"*Three Solutions.*—The problem seems to divide itself into three factors. The first is the factor of revising our street systems. That is a big order, of course, and not very many communities probably will feel that they can jump into it and do all that is necessary.

"The second phase is the matter of what may be called palliatives. By that I mean a remedy which does not entirely cure. The third phase is the matter of the police, traffic lights, and ordinances."

Revising Street Systems.—In the matter of replanning, one of the most difficult problems is that of the through traffic in a large sense superimposed on local traffic. Apparently, only large scale reconstruction is effective in many places, as in Chicago, Ill., Detroit, Mich., Jersey City, N. J., and New York, N. Y. Of course, there is the possibility of widening streets parallel to congested areas, thus creating a sort of by-pass. In details of design, Mr. Holleran mentioned widening pavements, proper lighting, and the elimination of grade intersections as desirable. Grade crossings cannot be avoided in business areas, but traffic islands can be substituted to good purpose. These islands should have a radius of at least 60 ft.

Communities which do not think they can go to these extremes, should recollect that wise expenditure is largely returned in resulting increased valuation. The experience of the Westchester County (New York) Park Commission, of which Mr. Holleran is Deputy Chief Engineer, is a striking proof of this.† He mentioned cases where the valuations have increased ten times since the parkways were started. Other communities have had the same experience where they had the courage to plan and execute such improvements.

* Deputy Chf. Engr., Westchester County Park Comm., Bronxville, N. Y.

† See "Westchester County Planning and the Park System," by Jay Downer, M. Am. Soc. C. E., before the City Planning Division, p.

"This is the province of the engineers. They are the ones that should lead and should think of what ought to be done and get it in shape so that they can 'sell it' to the people themselves. It is also the province of the public officials who have to put through the bond issues necessary for such construction. It costs a lot of money to do it. Engineers are obliged to lead and they should assume that burden."

Other Expedients.—If extensive re-planning is not feasible, certain "palliatives" are available. It is possible to cut, say, 3 ft., from each sidewalk and make a parking lane; parking can be prohibited entirely in certain congested districts; and curb radii at street intersections can be increased, where now the original curb designed for horse-drawn traffic still exists.

Another source of great danger, especially outside cities, is that pedestrians have no place to walk other than on the pavement. This is not only dangerous, but a positive impediment to traffic. The solution is obvious—paths or pedestrian walks should be provided. In addition, the pavement itself should be designed only for its particular traffic with special surfaces on steep grades. Then there is the matter of improper use of sidewlks and streets for storing of materials, digging of trenches, and improper repairs. All these have an effect in slowing up traffic or causing accidents. The resultant cures are of minor difficulty.

Police and Traffic Lights.—The third phase is one that the general public considers most important—traffic control. To Mr. Holleran's mind, this is greatly overdone in many communities. He mentioned one that uses at least a third more traffic lights than are needed; these impede rather than help traffic, as they halt the motorist at every turn. Another difficulty is the application of traffic control beyond its necessary limits, such as the use of a Sunday schedule during week days, when little or no traffic is to be handled.

It is possible, also, to have too many police, especially if they are too officious. Likewise, many ordinances that are unenforceable could be eliminated. Such, for example, is one limiting the maximum speed to 15 miles per hour. As examples, Mr. Holleran cited the Bronx River and Hutchinson River Parkways in Westchester County. There the police are supposed to help motorists in any way, but not to be too much in evidence, and the official allowable speed is 35 miles per hour. These are good proofs "that the efficiency of traffic regulation ought to be measured by the absence of usual control features."

DISCUSSION

TRAFFIC CONTROL

By E. W. James,* Assoc. M. Am. Soc. C. E.

The Future, the Important Test.—In discussing the general question, Mr. James expressed the viewpoint of Government road officials who "are interested

* Chf., Div. of Design, U. S. Bureau of Public Roads, Washington, D. C.

in traffic control only that we may be able to design our product, *i. e.*, highways and roads, so that they are in condition to be used 100% efficiently." As he stated,

"One of the most important tasks of the traffic engineer to-day is not confined merely to present traffic control, but should be concerned with very much larger matters which will insure that worse future conditions will not result from mistakes of the present.

"In the past, conditions have been deliberately created because their effects were not foreseen, and perhaps were not even foreseeable. But present experience seems a sufficient teacher that certain conditions are to be definitely avoided in the future if we are to prevent traffic congestion which will make efficient use of our cities practically impossible without enormously expensive methods of relief."

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"*Regional Planning as a Safeguard.*—If we are to control the traffic situation in the future as it must necessarily be controlled we must look to other means than those provided by the traffic squad, the automatic signal, or the corner policeman. Obviously, congestion occurs where traffic has to stop, not where it can keep moving. Interference with the physical flow of vehicles causes congestion and not mere numbers alone; and very generally such interference causes congestion where the number of vehicles is far below what the pavements will carry if traffic can be kept moving at a reasonable speed. One of the greatest sources of traffic interference is the existence of too frequent street intersections throughout urban and suburban areas.

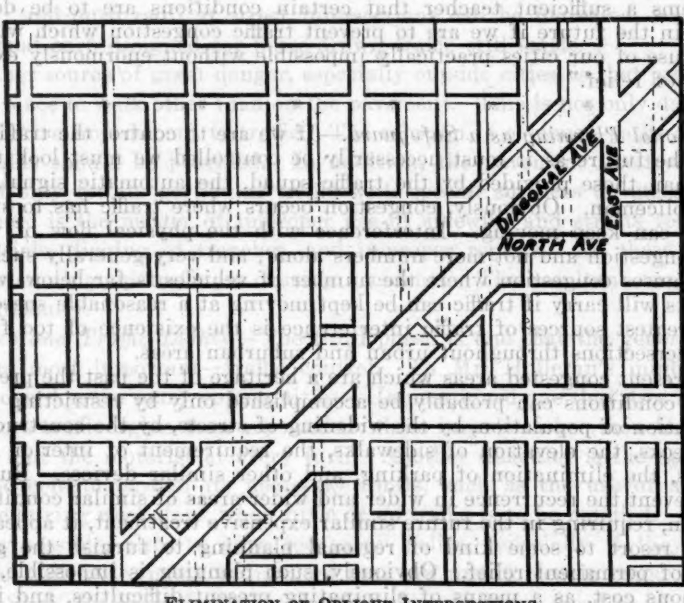
"In present congested areas which are a heritage of the past the prevention of worse conditions can probably be accomplished only by restricting further concentration of population, by the widening of streets, by the construction of second decks, the elevation of sidewalks, the requirement of interior loading platforms, the elimination of parking, and other similar devices. But if we are to prevent the recurrence in wider and wider areas of similar conditions of congestion, requiring in the future similar expensive treatment, it appears that we must resort to some kind of regional planning to furnish the greatest measure of permanent relief. Obviously, such planning is impossible, except at enormous cost, as a means of eliminating present difficulties, and it must be confined to the treatment of suburban and semi-rural sections to insure against the future creation of new urban areas which will become just as bad as those with which we are now struggling."

Other Measures of Relief.—As Mr. James pointed out, most State systems fail to reflect the needs of traffic. For example, a traffic flow map eloquently points the needs of increasing capacity toward centers of population. Certainly this is hardly true in practice. If, however, additional widths cannot be supplied, the condition of the pavement and the frequency of interruptions to the flow traffic should become more and more favorable toward centers of converging traffic. As an actual fact, the opposite occurs—"there is not only greater restriction of traffic due to lack of width, but greater interruption to the free movement of vehicles."

On the principle that diagonal street intersections hinder traffic, Mr. James noted that Major F. S. Bessen had suggested an improvement for Washington, D. C., by reducing the interference of diagonal streets, and substituting rectangular intersections. The diagram illustrates the arrangement. Mr. James stated his faith in the rotary intersection, also well illustrated in Washington. He pointed out, however, that even this island type was subject to some

danger, and frequently requires that lights be installed or that a supplementary safety zone be used. As he stated,

"The relative conditions of traffic and radii of the central parking area and of the central pavement, and the number of radial streets, and the particular paths of traffic, whether the particular path is directly through the circle for instance, in at the north or out at the south or whether it comes in and goes out at angles, may have effects on the efficiency of the circle that we know nothing about whatever."

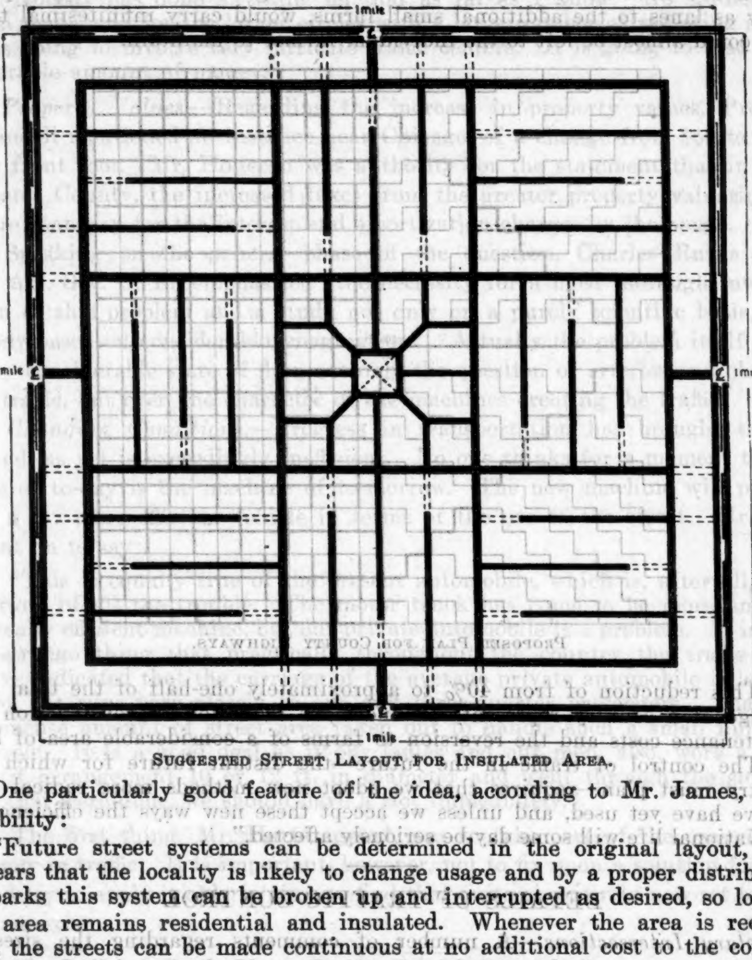


ELIMINATION OF OBLIQUE INTERSECTIONS.

Insulated Areas, A Recent Proposal.—Obviously, it is impossible, Mr. James considered, to eliminate dangerous intersections entirely. He mentioned one arrangement, however, styled "insulated areas", for relieving dangerous situations. This consisted essentially of the street arrangement within, say, a square mile, whereby there would be no incentive to through traffic, although there would be every means for interior communication. Only four means of exit and entrance would be available, and the streets themselves could be of narrow width, since they would handle only local traffic.

If necessary, the subdivision of such an area could be made to provide (as shown on the plan) for future extensions of through routes by cutting streets through present parks and by now providing wide rights of way for further enlargement of pavements. Mr. James estimated for such a subdivision that with "eight householders to the acre, all the traffic could leave the area on its way to work within an hour without street congestion at the exits." The interruptions by means of parks would not deter foot traffic, which

could use the parkways to be provided. Such a sub-development is Radburn, near Paterson, N. J. Although the partly insulated area is only one-eighth of a square mile, it would have just about the density of population mentioned.



SUGGESTED STREET LAYOUT FOR INSULATED AREA.

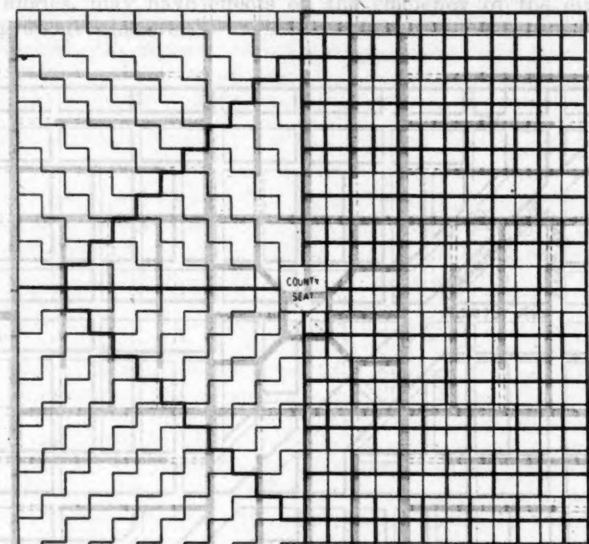
One particularly good feature of the idea, according to Mr. James, is its flexibility:

"Future street systems can be determined in the original layout, if it appears that the locality is likely to change usage and by a proper distribution of parks this system can be broken up and interrupted as desired, so long as the area remains residential and insulated. Whenever the area is reclassified, the streets can be made continuous at no additional cost to the community for rights of way by destroying a part of the parks."

Improving County Road Systems.—Applying the principles to rural highways, Mr. James presented a diagram:

"To show what might be done in the way of county planning of highways in a typical county now laid out with 2 miles of road or more per sq. mile according to the official section layout of the old public land States. Such layouts would obviously require variations to give access to sub-centers, such as railroad shipping points, other than the county seat, and no one standard plan could be applied; but some possible designs are shown. Distances to the county seat are little changed, and distances between scattered points in the county would be considered negligible if the substantial possible reduction of total mileage resulted in the improvement of completed local road systems.

"The layouts illustrated represent reductions in total road mileage of 43, 38, and 46%, respectively. These layouts assume that the average farm is a quarter-section, which fact holds for Iowa, and which is not far wrong for Illinois and Wisconsin. Additional mileage would be required where the farms are smaller. But such additional mileage could be arranged practically as lanes to the additional small farms, would carry infinitesimal traffic, and could almost be left to the individual farmer to maintain.



PROPOSED PLAN FOR COUNTY HIGHWAYS.

"This reduction of from 40% to approximately one-half of the total road mileage of a country represents an enormous reduction in construction and maintenance costs and the reversion to farms of a considerable area of land.

"The control of traffic in the future—the distant future for which this country must plan—requires that we adopt new methods more radical than any we have yet used, and unless we accept these new ways the efficiency of our National life will some day be seriously affected."

DETAILS OF TRAFFIC CONTROL

Rotary Intersections.—A number of comments regarding the sizes of rotary intersections were offered by R. W. Crum, J. S. Crandell, L. G. Holleran, Members, Am. Soc. C. E., and E. W. James, Assoc. M. Am. Soc. C. E. The 60-ft. diameter was defended on the ground of agreement with the minimum turning radius of large cars. On the other hand, to enlarge the size would improve conditions. According to Mr. James, a 40-ft. circle is used in Richmond, Va., and one of at least 133 ft. in diameter in Washington. He estimated that the width of the circular pavement should equal 25% of the aggregate width of all radiating roadways:

"What is required is sufficient width of pavement and sufficient distances between spokes to permit of the necessary intervening traffic in the circle.

Some traffic will have to pass from an interior lane on the circle to an exterior lane to get on to one of the spokes. There must be room for that in width and in distance between spokes, so that affects both the width of your pavement and your radius, of course.

"Nobody has done anything on that as far as I know. No studies have been made, and for the larger circles with a large number of radiating streets, it is going to involve very intricate traffic centers. It is going to cost a considerable amount of money."

Property Values.—Regarding the increase in property values, Professor Crandell mentioned an instance near Chicago, of a change from $50 to $1 000 per front foot. Mr. Holleran was authority for the statement that in Westchester County, the increased taxes from the greater property valuation will more than pay for the interest and amortization charges on the bonds.

Speaking on the general phase of the question, Charles Rufus Harte, M. Am. Soc. C. E., emphasized "the necessity for a most thorough investigation of this problem and a study not only on a purely scientific basis, but a study based on considerable imagination". Actually the problem itself is in a very considerable state of flux, not only the question of arteries and character of traffic, but even the character of the machines creating the traffic.

Changing Conditions.—Progress in transportation has brought the bus, which as yet is exceedingly inefficient. No one thinks for a moment that the bus of to-day is the machine of to-morrow. The new machine will probably be a far more efficient vehicle in terms of the use of the street. Mr. Harte went on to say:

"This is equally true of the present automobile, which is, after all, at the bottom of all the trouble. The motor truck has come to be more and more a really efficient machine, but our private automobile is a problem. It is rather a strange thing that practically throughout the country the traffic checks have indicated that the carriage of the average private automobile is less than two. It runs very closely to one and three-quarter passengers. That is an immense amount of street area taken out to handle such a small number of people. It is a good deal as if our ladies and our men, also, wore a hooped skirt arrangement 10 or 12 ft. in diameter and went through the sidewalks. If that were done, we should have a riot immediately."

The first thing, Mr. Harte agreed, was to take care of the present emergency in traffic. It is important, however, not to fix upon a solution for to-day and try to apply it only in time to find that quite an entirely new set of conditions exist.

RESPONSIBILITY OF THE HIGHWAY ENGINEER TO THE PUBLIC

By A. R. Hirst,* M. Am. Soc. C. E.

Two Kinds of Engineer.—This subject also attracted much attention. According to Mr. Hirst,

"Highway engineers may be divided broadly into two classes. The first, and probably the largest class, is the 'Alice sit by the fire' class so common in all walks of life, especially in the public service. Highway engineers in

* Cons. Engr., Madison, Wis.

this class have a job, thank Heaven for it, and will not risk losing it by the exercise of brain or person which runs counter to the 'God of Things as they are'. Their motto is 'let George do it' and when 'George' has done it and the world approves it as good, our 'Alice' follows—a very long way behind.

"This class are not much more than glorified office boys. They take orders from some one higher up; shut their eyes to everything they are not desired to see; and dodge every decision that can possibly be passed either up or down the line. If highway engineering progress had depended upon them, we would still be building dirt roads with shovels and traveling them behind quadrupeds.

"The second broad class of highway engineers have power, and exercise it; they are more or less free to function, and are glad to function; they have responsibilities and choices, and are glad of it. These men build not only highways, bridges, and structures, but conditions, public sentiment, legislation, organization, and men. Unless a highway engineer is a real builder his name is a misnomer.

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"The real function of a highway engineer—National, State, county, urban, or local—is to give to his clients, the people, the best highway service every day in the year that the funds available permit, at the lowest possible total cost over the years. With the constantly vastly growing fleets of motor vehicles this has been and is a tremendous problem."

Solving Financial Problems.—So many are the outlets for public funds, Mr. Hirst considered, that highway departments have suffered. The sources of revenue are, in general, the license fee and the gas tax. These, in turn, have to be subdivided between county, city, and local units. Unfortunately, the highway engineer has had to lead in developing State highway revenues. He has had little help from the automobile manufacturer, whereas the public roads have made the manufacturers prosperous. One of his great financial difficulties is in dealing with regulating boards, such as legislatures, city councils, and county boards. Thus, one of the most important of his responsibilities is to induce the public to spend more money on highways than its natural tendency would provide.

Again, a similar difficulty is in dividing the available funds between construction and maintenance for best total effect. His fight is to keep the legislative budget properly balanced. Any cut must be in construction rather than in maintenance, but as construction is eliminated, maintenance inevitably increases. Further still he has to guard the public from "pork barrel" tactics. No one else can do this and he must not dodge.

Real Cost, in Use, of Highways.—Having escaped all these pitfalls, the highway engineer's simplest job begins. Let him but have a competent engineer of maintenance to whom he can turn over the necessary funds and that part of the work is readily accomplished by more or less standardized methods. Construction, however, is more directly a personal problem. It involves choice of location, width, and type of surfacing. Here he saves or loses money for his public, not so much in building wrong surfaces, as in selecting wrong routes. So large is the present traffic, that first cost is never a material factor; rather economy of distance and freedom from grade crossings and curves may become the controlling factors.

That roads may have been built too well and cost too much is the smallest of the sins on the heads of highway engineers, according to Mr. Hirst. Whereas dollars may have been wasted thus, thousands of dollars have been squandered in "under-planning and improper locations". The real cost of highways is not the first cost, but the cost of using them. He mentioned as an example Wisconsin where he was formerly State Highway Engineer. The State has 80 000 miles of highways, and 800 000 motor vehicles; it spends $30 000 000 per year on highways, and $300 000 000 on automobiles.

"The savings possible on the large amount are probably several times those possible on the small. The small one comes directly out of taxes, the large one out of our other pocketbook. We recognize one because it is all assembled, known, and published. The other is distributed among possibly 700 000 owners, probably not 10% of whom know even what it costs to own and operate their own motor vehicles.

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"It is a real matter for debate whether it is not better to spend the available funds for the 'must be' length, building 20 ft. wide to the depth financially possible rather than building an 18 or even a 16-ft. width of super-depth. A few slabs or sections of the wide thin road may crack or fail, but that usually only costs money for repairs, while the too narrow road inevitably costs the wreckage of bodies and lives and motor vehicles.

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"*Engineers and Patents.*—One of the strange manifestations in highway work has been the general reaction of highway officials against patented pavements and bridges. The minute an engineer gets a patent on a new worth while idea, apparently the whole engineering world considers him as a criminal, or at least unethical, and proceeds to 'razz' his invention, and use something just close enough to it to avoid infringement. This attitude on the part of engineers has been a great deterrent to highway progress. There has been little encouragement to think or work on ideas or to patent pavements or bridges because the unfriendly attitude of engineers makes it almost impossible to market any new idea with profit.

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"The competent highway engineer can serve his people by inventing and by encouraging inventions, trying them out on a small scale, and helping to develop them if they are good and to kill them if they cannot justify themselves in service by results.

"*Improving Concrete Pavements.*—At present and for some years the thoughts of highway engineers and all allied interests have been largely concentrated on improving concrete roads. Much progress has been made, but one has only to drive and keep his eyes open, to be convinced that the concrete surface has still a long way to go before it delivers the maximum service for its cost.

"It is universally considered that concrete must be cured, that is, the concrete surface. I do not know of an engineering department in America that cures a concrete base.

"A major responsibility of the highway engineer spending millions on concrete surfaces and bases is to know something of the modern theory and practice of building concrete of maximum strength at minimum cost. Outside of so few, that it's startling, our profession really knows astoundingly little about concrete, and practices even less than it knows.

"Concrete as now built is not perfect and the sooner we all recognize this and abandon our attitude of smug complacency and search for improved

methods of sizing, proportioning, mixing, placing, compacting, finishing, and curing concrete surfaces and bases the better for the cement industry, the engineers, and the public pocketbook. The last dog is not hung yet as far as producing 100% concrete is concerned. We now may be averaging 75% of the results possible at the same cost.

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"Pitfalls of Inspection.—Politicians have always considered the inspection of public work as their 'white meat.' Said inspection is, in general, performed by 'the lame, the halt, and the blind,' the time server, and the 'ne'er do well'. The best inspector, in the eyes of the average politician, is the man who controls the most votes.

"Far too many engineers content themselves with drawing a pretty picture of what the job should be, then writing specifications which they themselves do not understand. After that is done they leave the actual construction on the 'knees of the Gods' and in the hands of 'Bill Knownothing', the inspector. A good highway engineer cannot dodge the responsibility of really seeing that the job is built, practically as specified and is paid for; it's a hard fight for a worth while cause. Insisting on proper inspection may possibly cost the engineer his job, but that will seldom happen.

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"The engineer has a responsibility to his helpers—the contractors on public work. He can put sensible, early payment clauses in his specifications and he can see that they are honored in the observance and not in the breach. Highway contracting is a tough enough game at best without complicating it by making an 'Endowment at Age 85' policy out of the payments.

"Here the engineer can seldom 'pass the buck' to any one else. Where the engineer wishes it, payments are prompt (where the laws permit). Where the laws interfere they can be changed, if the engineer cares a 'whoop' to change them.

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"Problems of Personnel.—The engineer who dies or leaves his job with no one in the organization competent to succeed him and do as well or better than he, has not served his public. He can only train his proper successor by developing a sense of initiative and responsibility clear down the line.

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"The trouble with highway engineering is that it is public engineering. It must be carried on and developed in the field of American politics. As the highway programs have become larger the organizations have become more attractive to the politicians. This has, unfortunately, been most true in the State highway field where the opportunity is larger and the chance of doing worth while things the best. Here change is the unceasing order of the day.

"Where, if anywhere, permanency should rule as long as good progress is made, the job of being a State highway engineer is just about as stable as wet mud. Without the news of yesterday's changes, there are to-day, we believe, about fifteen heads of State highway departments who were the heads of the same departments four years ago.

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"The Engineer of the Future.—Fortunately, county and local highway units are much more stable. Apparently it is to the highway engineers in the lower ranks of State highway departments and in the counties that we must look for that permanency which only can lead to constructive achievement The engineers farther down the line have been too long resting on their oars, expecting the boat to be pulled and all the progress to be made by the chiefs of the State highway departments. They will have to start to row, because

the old stroke oars are getting such a dose of political 'knock-out drops' that the remainder of the crew will have to get into action if the fair rate of highway progress which has so far been kept up, is not going to slow down to a crawl.

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"Let no one think the job of building American highways is well toward completion. It has just started. There is room and time for a hundred geniuses to come up out of the 'bushes' and 'strut their stuff'. But one cannot be a genius or he cannot strut if he is hide-bound, atrophied, or paralyzed. His body must be alive and his brain alert; he must be a fighter, and love it; he must be a philanthropist, and look for a bare living; he must love work and love people; he must be a diplomat, and live it; he must be a Man as well as an Engineer. The field is nation wide and world wide. Who are going to be the Macadams, Telfords, Brindleys, Stephensons, Bessemers, Watts of the very modern highway 'game'?"

DISCUSSION

HIGHWAY ENGINEERS' RESPONSIBILITY

By W. H. Connell,* M. Am. Soc. C. E.

Public Service.—Agreeing with Mr. Hirst in his fundamental ideas, W. H. Connell, M. Am. Soc. C. E., stated that it applied just as much to any other type of engineer. By comparison he claimed that consulting engineering was much simpler than public service, because responsibility ceases after the presentation of plans. His experience led him to believe that under this system of government there was no great help for present conditions. Minor improvements have occurred and continue to take place, but many engineers in public service, through the very nature of their jobs, will have to content themselves with doing the best they can and trying to improve conditions through persuasion rather than through a militant leadership.

Purifying public service seems to go in cycles; it is not steady nor along a straight line. After considerable advance, certain interests may get control and retard progress, but they can never obliterate the good that has been done. Because of poor inspection alone, certain localities are only getting fifty cents worth of work for each dollar. Out of $2 000 000 000 yearly expenditure on highways, $400 000 000 is being wasted through inefficiency. Specifications have been improved until they are of the best. The fault is with the actual supervision.

Good Lieutenants.—To correct conditions, Mr. Connell insisted it is necessary to support the reformers who have sufficient experience. The gist of the matter is to inform the public and educate the politicians. Mr. Connell went on,

"Practically all the engineers I have observed in public service are doing a great deal of good, although they are not all active leaders. We must not

* Executive Director, Tri-State Regional Planning Federation, Philadelphia, Pa.

lose sight of two facts: For one thing they are doing a lot of good and we should help them to bring about the conditions they would like; in the second place, there are as many good lieutenants as there are leaders."

Taxes and Patents.—Remarking that in the matter of revenue the richer communities had to help the poorer, he pointed out that one solution for cities was to assess for itself a vehicle tax. Mr. Connell explained some of the money loss in locating and designing as due to the rush of construction. This, in part, is being obviated by the formulation of ultimate State road plans. Still another opportunity for waste is in maintenance. After a great deal of sad experience, the State of Pennsylvania had decided that superintendents of maintenance and everybody in the Highway Department should be absolutely free from any political endorsement.

His views on patented articles and methods were that most machinery is patented, but that there are cases of equally available machines. Likewise with pavement, there is no objection to the patented pavement, provided there is competition with other pavements. As a matter of fact, the antagonism to patented pavements has somewhat stimulated them through advertising. He closed his discussion with a note of optimism. Throughout a few years, definite progress had been made. Although conditions now are not what they should be, there has been a very marked improvement. In general, city engineers need more help than State officials.

CITY FUNDS AND STATE AID

Towns Need Help.—In opening the general discussion, Harry Tucker, M. Am. Soc. C. E., alluded particularly to the financing of road work in the rural communities of the South. There, many State highway systems have been organized only within the last five or six years. A gasoline tax is available, it is true. With State highways came also county highways and then the public school systems became greatly enlarged so that the consequent increases in taxes in the counties and small towns made them "practically confiscatory". Professor Tucker agreed that the State highway tax might well be applied to State highways until the system had been completed. Thereafter, the problem would be as to how it could be divided equitably among counties, cities, and towns.

Even at present, for example, Virginia gives a part of the revenue back to the counties; Alabama is said to devote 1 cent of its gasoline tax to the incorporated towns and cities. In still another variation, North Carolina is considering putting all the county highways in the State highway system. He advised that highway engineers give serious thought to this problem so that when the question comes up in their own States, as it certainly will, they will have a definite proposition to submit.

Traffic Its Own Worst Enemy.—In reply to this suggestion, Mr. Hirst took the ground that the counties would never get the State money, because the moment the State system seems to be finished, it has to be rebuilt or enlarged. Westchester County, in New York State, has already spent

$50 000 000, mostly on highways, and will probably spend in all $100 000 000, all without help from the automobile tax. Around New York City for bridges and tunnels, approximately $200 000 000 is being spent, all to be paid for by the motor vehicle owners using the improvements. It becomes a question of just how long taxpayers with all their generosity are going to be able to support these developments for the use of motor vehicles. Apparently, automobile manufacturers do not realize that property in general has about reached its limit in providing for transportation, with traffic lights, special policemen, and State betterments.

Traffic simply creates traffic. Mr. Hirst drew examples from New York City traffic over the Hudson River. As long as people had to use ferries, this handicap minimized traffic; but with the new bridges and tunnels the further congestion in New York City will be enormous. The Holland Tunnel was built to be self-supporting; but the State of New Jersey has had to spend $30 000 000 on its approaches.

Financing Problems.—Highway financing is the crux of the whole situation because it bears on the States themselves and on their cities, villages, and townships. One essential is to separate grade crossings on major highways, which are certainly as important as railroad crossings.

In this connection, Mr. Holleran was authority for the statement that grade-crossing separations usually cost less than $100 000. On his Westchester work, for 60-ft. parkways, it has been as low as $55 000.

The question of financing also appealed as of primary importance to C. E. Myers, M. Am. Soc. C. E. The situation in Philadelphia, Pa., he took as more or less typical. Although having 20% of the State's population, it received only about 1.5% of the gas tax. Even then, not all the money went into highway work, as the City Council used some of it for other purposes. One advantage at least of the State expenditure, Mr. Myers thought, was that all of it went into highway work where it belonged. He believed that the one great problem was to apportion some of the funds to city work so that it could not be handled politically and thus taken away from State work. He advocated a State aid for cities somewhat similar to Government aid for States.

In this connection, Mr. James reminded the meeting that in Virginia many towns are included in the State highway routes, and so share in the improvement. Still more extreme is the condition in Maryland, where Baltimore contributes 75% of all the State taxes. There the main routes through the city are practically in the hands of the State Highway Department.

A sidelight from the reverse of this picture was presented by H. K. Bishop, Chairman of the Division, and Presiding Officer, who observed that censuses indicated 75 to 80% of the traffic carried on the State highway systems, the bulk of this coming from cities. Thus, the cities actually do obtain a great deal of service from the State highway systems in repayment for their taxes.

Advantages of Road Program.—The final discussion of the session was contributed by F. J. Mulvihill, Assoc. M. Am. Soc. C. E., who observed that the attitude on any controversy between the State and the towns altogether depended on the point of view. He was favorable to the use of patented

machinery because of the resulting speedy construction without sacrificing to quality.

Economically, there results an enormous saving in interest alone, not to mention the social advantage of having the improvements more quickly available for use. He even looked forward to the day when pavements could be factory-made under ideal conditions and laid with greater rapidity. Mr. Mulvihill drew a lesson from the old fable in which the monkey thrust his paw into a jar and then could not extract it with all the nuts he had clutched. He believed that by constructing piecemeal instead of in co-ordination with an ultimate plan based on a proper program of development, it is not possible to get any of the "nuts" out of the jar. By working at a definite plan and program he considered that each nut would come out in its own time, so that eventually the jar would be empty and the people would have the nuts.

STATUS AND PROGRESS IN THE ART OF HIGHWAY ENGINEERING

REPORT OF THE EXECUTIVE COMMITTEE

The object of this report is to present concisely the outstanding developments in the art of highway engineering. Due to its brevity, detail as to methods or specifications used in practice cannot be presented. It is hoped that the preparation of the report may be justified, in that it will call to attention certain features of the art worthy of further study and research.

Foundations and Drainage.—Adequate foundations and proper drainage must be provided, no matter what type of surface is being constructed, if good results are to be obtained. As the demands of traffic have necessitated the construction of expensive roads, the importance of foundations and drainage is being given more consideration now than in the past.

Earth Roads.—The term, "earth roads", as generally accepted, includes all types constructed of natural soil. The variable nature of the many soils used, of necessity demand different methods of construction and maintenance, but for the purpose of this report they are grouped together. The recent developments in the methods of construction and maintenance are, in general, applicable to all types.

The greatest advances have come through a more complete knowledge and appreciation of the effects of adequate drainage on future maintenance costs and ultimate life. The modern tendency in design is toward higher grades across low ground and deeper ditches through cuts and along shallow fills, in order to keep the road surface well above the ground-water level at all times. In certain localities, however, where deep ditches prevail, break-ups in the spring often appear worse than where shallow ditches are used. In the Middle West there has been a marked increase in the amount of tile used in sub-surface drainage as a ready means of quick disposal of surface water.

The amount of crown for earth roads has been materially reduced in recent years. It is now quite common practice to construct a flat section, depending on shoulder settlement and future maintenance to provide such crown as is desired. Crown heights of less than 6 in., even on important highways requiring wide roadways, are frequent.

More attention is being given to the details of constructing the roadway shoulder and to the back-sloping of cuts. Care in these details not only improves the appearance of the completed construction, but also results in lower future maintenance costs.

There has been a decided tendency toward the use of flat slopes between the road shoulder and the side ditch. Slopes of 3 in. to the foot are common and contribute greatly to the safety of motor vehicles.

The use of a thin layer of crushed stone of small size, which is maintained by frequent blading, and the addition of new stone as required, has met with excellent results at a low cost and has provided a satisfactory surface for

traffic up to 500 vehicles per day. Several thousand miles of State highways have been so improved. Surface treatments of bituminous materials or calcium chloride would make it practicable to carry a somewhat larger traffic without difficulty. A more general use of this method would add many miles of road that would be serviceable for most of the year.

The use of heavy units of equipment in the maintenance of earth roads has made it possible to keep the side ditches open at all times and to maintain the road section true to form. Light units of equipment are still used with success for surface maintenance. The relations between the selection of the proper type of equipment, the time of use, and the soil characteristics are receiving more attention from maintenance organizations.

The application of oil to earth road surfaces has met with favor in some States and is giving beneficial results in maintenance. Frequent light coats are, in general, more satisfactory than heavy applications at infrequent intervals. Little permanent improvement is obtained by the use of oil.

Gravel Roads.—The "feather edge" method of construction, whereby light applications of gravel are spread over the road surface and compacted under traffic, is being used to a large extent in certain parts of the country, while in other sections the "trench" method is still in favor. Greater care is being exercised in the selection of properly graded material containing sufficient binder to compact the mass under traffic, and in the removal of over-sized stone before depositing the material on the road.

Successful maintenance of gravel roads under other than very light traffic requires almost continuous working of the surface to keep it in smooth condition for travel. As a surface treatment the use of light oils and tars, and, in some localities, of calcium chloride, is common.

In the West fine crushed rock and gravel surfaces are being successfully treated by what is called the "processing method", in which an oil to 70% asphaltic content is incorporated with the surface. The old road is scarified to a depth of 2 or 3 in. and three applications of oil are made, using about ½ gal. per sq. yd. The road is thoroughly bladed and harrowed after each application to insure a complete mixing. The proper amount of oil to use is determined by a stain test and is varied according to the character of the surfacing material.

Water-Bound Macadam Roads.—No marked change in this type of construction has taken place. The use of water-bound macadam, except as a temporary expedient, is being abandoned. It is a question of economy whether, under certain traffic conditions, the use of this type with its expensive maintenance is warranted.

Bituminous Macadam Pavements.—The prevailing practice in the construction of bituminous macadam pavements calls for large stone in the wearing surface, usually of a size which will pass over a ring 1¼ in. in diameter and through a ring 2½ in. in diameter. The importance of stone of good quality is emphasized. However, one with a coefficient of wear as low as 6 may be successfully used if proper attention is given to its sizing. On heavy traffic roads, the wearing surface is usually 2½ to 3 in. thick after rolling. Whether the mosaic or closed type is the better is still a debatable question.

For the base course the sizes of stone are variable, uniformity not being so important as for the wearing surface; also stone of softer quality may be used. The depth varies, depending on the foundation upon which it is placed and on the traffic conditions. Where a depth of 2 and 3 in. might be used over a stone-fill sub-base, one of 4 to 6 in. would be proper over a gravel sub-base, or good earth sub-grade. Sand is more generally used for filling the base course than stone screenings. Base courses treated with bituminous material are not general.

Asphalt is more common than tar in this type of construction and oil asphalt predominates. A greater part of the work of application is now done by truck distributors, some of which have been developed to a high state of efficiency. The selection of the proper bituminous material with relation to the climatic and traffic conditions has an important bearing on the success of this type of construction.

Bituminous Concrete Pavements.—The construction of bituminous concrete pavements is being carried on without any marked changes. In lieu of cement concrete or other types a bituminous base is being used to a small extent.

The development of central-mixing plants where the bituminous concrete is prepared for either rail or truck shipment is rapidly progessing. In certain sections of the country both asphalt and tar mixtures may be obtained. Two types of mixture are furnished, one being prepared with a coarse, and the other with a fine, aggregate.

Much of this material is used in maintenance work and for re-surfacing worn-out pavements. The fact that the materials may be manipulated cold makes possible the use of bituminous concrete in places where otherwise the expense of erecting and operating a plant would be prohibitive.

Sheet Asphalt Pavements.—The diminution in production in the Mexican oilfields and the development of large fields in South America indicates that before long South American crude oil will be placed on the market. It is reported that considerable research work and experimentation is now being carried on in connection with the refining methods to be used with these oils.

To secure a more rational understanding of the design of asphalt paving mixtures, researches and tests are being conducted on the paving mixtures themselves. The stability test devised by The Asphalt Association is being used for measuring the resistance to displacement of these mixtures. Detailed knowledge of the characteristics of the individual constituents of the mix seems to be of less importance, from a practical standpoint, than knowledge of the mixtures made from these constituents; in other words, less dependence is being placed upon the usual specified grading requirements for sand than ever before. The new stability test is very valuable, if considered in connection with the density of the paving mixture as compressed on the street, considering also the percentage of voids and the workability. Interesting results are produced by other methods of measuring resistance to displacement of both fine and coarse-aggregate paving mixtures.

Mechanical spreading and raking devices for asphalt paving mixtures are being developed and considerable progress has been made along this line within the past two years. One company has modified its concrete finishing machine to make it adaptable for finishing asphalt pavements. Another machine spreads and combs the mixtures just before rolling. Greater attention is being paid to securing uniformity of contour. The importance of this detail of construction cannot be emphasized too greatly. There is no reason why asphalt pavements should not be as smooth as any other type, and engineers should insist upon uniform contour. Greater attention is being paid to securing the maximum density from any mixture, and the density requirement is coming more and more into general use in asphalt specifications. More exact control of the uniformity of plant output is desirable, and investigations are being conducted to determine the best possible means of obtaining and keeping such control, with the thought always in mind to reduce the cost of operation where possible.

Vitrified Block Pavements.—The more recent developments in vitrified block paving are: (1) The use of a thinner block; (2) the use of asphalt filler; and (3) the elimination of the cement from the sand bed and the reduction in depth of the bed.

For no particular reason, the original vitrified block pavements in America were 4 in. in depth. About 1915, engineers began to consider the use of a block of less depth. In 1925 the U. S. Bureau of Public Roads made a test at Washington, D. C., of vitrified blocks having a depth of 2 in., 2½ in., 3 in., 3½ in., and 4 in. They state in their report that the 2½-in. block is ample to carry the heavier types of traffic.

At one time vitrified block pavement on a concrete base was considered to be monolithic, and an effort was made to have it so. Now, however, the blocks are considered to be a wearing surface. Formerly, cement grout was the standard filler; at present, the best practice is to use a bituminous filler with simply a sand bed of a depth not exceeding ¾ in.

Stone Block Pavements.—The principal development in stone block in the past few years has been to reduce the depth of the block, in an effort to reduce the price per square yard. A number of bridges have been paved with shallow block having an average depth of 3½ in., with an allowable variation of from 3⅜ to 3⅝ in.

The length in some cases has been increased to a size of from 8 to 11 in., with a width of from 4 to 5 in. These blocks are somewhat wider than they are deep, and similar to what is commonly known as the "resurfacing" block. This method of cutting produces a better surface on the block, because splitting the granite the easiest way of the stone and using this surface for the head gives naturally fewer depressions in the surface.

The present standard block has a width of 3½ to 4½ in., but an alternate much in use has a width of 4½ to 5½ in., a length of 8 to 12 in., and a depth of 4¾ to 5¼ in. Bituminous fillers seem to be coming into general practice rather than cement grout, and as in the case of the vitrified block pavements, the cement in the bed is eliminated when a bituminous filler is used.

Cement Concrete Pavements.—The construction of cement concrete pavements is gradually approaching a standard practice, but it will be some time before practice will be uniform in all its details.

Much research work has been done in late years on the selection and grading of the materials and the proportions to be used. The most recent outstanding feature along these lines is the determination of the proper mix by the water-cement-ratio theory. That this theory is a practical one, there can be no question. It has been incorporated in the building codes of several municipalities and in the specifications of some State highway departments. Proportioning concrete by this theory should lead to cheaper concrete of a strength equal to that now obtained by arbitrarily fixing the proportions of the mix. The weighing of the aggregates makes a more accurate proportioning of the mix than the volume method. The time is probably not far distant when the weighing method will be generally adopted.

In the construction of concrete pavements, early strength is a desirable feature in order to speed up construction and hamper traffic as little as possible. Some high early-strength cements that accomplish this purpose are no more expensive than the ordinary cements. The use of calcium chloride in solution with the mixing water is also good practice to secure an early strength.

Field tests are essential in following up the construction of a concrete pavement. Grading tests, slump tests, and beam tests may be made in the field and a careful control of the work obtained. These tests can be supplemented by tests in the laboratory on cores taken from the pavement or cylinders cast on the job.

Reinforced pavements are now considered the best practice, but the type and method of reinforcement have changed. The grid-bar type and marginal bar type each have their proponents and there are also those who favor the mesh type or a combination of the bar and mesh types. It is too early yet to say which method is the best. There is also a difference of opinion as to whether dowels on the longitudinal joints are essential. The transverse joints, however, are almost always doweled.

Methods of curing are quite variable. It is a question of economy and of traffic requirements whether an accelerator be used or whether resort be had to dirt, hay, or ponding.

To facilitate construction and prevent hindrance to traffic, it is common practice to construct concrete pavements in strips. For instance, a 20-ft. pavement would be built in two 10-ft. widths, or if 40 ft. is required, four 10-ft. widths would be built. A longitudinal joint does not seem to be a serious drawback, although it may require some maintenance. The practice of placing a bituminous filler on the longitudinal joint is not uniform, whereas in the transverse joints it is always required. Great care must be taken in finishing the pavement at the transverse joints, in order to avoid bumps in the surface and weak edges at the joints that will have a tendency to spall.

Depths have gradually increased and very few pavements are now built less than 7 in. thick; under heavy traffic a depth of 9 in. may be warranted. While both the uniform-depth and the thickened-edge types are used, the

thickened edge predominates. The old practice of building the pavements thicker in the center than at the sides has been discarded.

Equipment used in the construction of concrete pavements is being constantly improved. Measuring bins, by means of which the aggregates may be accurately proportioned, are essential. Both volume and weight types of bins may be had. Mixers should be kept in good repair and should be equipped with timing devices and with accurate water-measuring devices. Mechanical finishing is more economical than hand methods and generally yields better results. Steel forms are almost exclusively used. Methods of handling the aggregate to the mixer vary, depending on job conditions and sources of supply. It is common practice, however, to require that accurately measured batches be delivered to the mixer ready to dump into the skip.

One-course pavements are practically a standard of construction. A sheet method of construction has been tried, in which a rich thin top is laid on a base course of a leaner mix. A fabric is placed between the two courses to provide a cleavage plane for the purpose of facilitating repairs and replacements.

Traffic.—Traffic conditions are constantly changing and presenting problems both interesting and new. More attention is being given to alignment than ever before. Pavement widths on State highways as great as 40 ft. are not unusual. Scenic trails are being constructed primarily for the enjoyment of the motorist. The elimination of grade crossings, bad bridge approaches, under-passes, and other points of danger, is being undertaken in some localities, and at great expense, in order to provide safer travel. The use of automatic signal lights at dangerous points, frequent markers, road signs, and other appurtenances to promote safety and comfort has become the rule rather than the exception.

Municipalities have their problems, among which those of parking and regulation of flow of traffic within the congested areas are outstanding. The routing of through traffic around the cities is now being given more attention than formerly. The demand for highways for winter travel has led to the adoption of some plan of snow removal practically everywhere within the snow-belt. Large expenditures and well organized forces enable some departments to keep the main highways open to traffic even under extreme winter conditions.

Conclusion.—The art of highway engineering is still in a process of development. The ever-increasing demands of traffic are necessitating new methods or modifications of old ones. More thorough study and research of these problems is leading in the right direction and undoubtedly there is a steady and gradual improvement in the methods and types of construction.

H. K. Bishop, *Chairman,*
John H. Ames,
H. Eltinge Breed,
Henry B. Drowne,
C. E. Myers,
C. D. Curtiss, *Secretary,*
Executive Committee.

SANITARY ENGINEERING DIVISION

JANUARY 17, 1929—10:00 A. M. TO 1:00 P. M.

After brief remarks by Robert Spurr Weston, M. Am. Soc. C. E., Chairman of the Division, the meeting launched into consideration of the main topic—The Sewerage Development in Milwaukee, Wis., and especially its new Activated Sludge Sewage Disposal Plant. This was presented by Darwin W. Townsend, M. Am. Soc. C. E., who chose the most important points from his more complete paper to give a comprehensive view of this installation.

MILWAUKEE SEWAGE DISPOSAL PLANT

By Darwin W. Townsend,* M. Am. Soc. C. E.

The Original Problem.—The re-vamping of the sanitary sewage disposal work at Milwaukee began through the organization of a Sewage Commission in September, 1913. Its duties have been mainly in the interest of eliminating sewage pollution from three rivers which flow through the city, and into Lake Michigan, the source of the city's water supply. In general, this work consisted of a new collecting system—intercepting sewers, inverted siphons, etc.—and a modern sewage disposal plant, which was eventually built on the shore of the lake.

Long since, the reduction in the oxygen supply of the rivers, especially during the summer, had created an intolerable condition of pollution. The primary aim, therefore, was to improve the sewage treatment to yield an effluent of high standard. In connection with this, also, the question of disposing of the sludge was considered important; recently it has become a still more prominent factor. In all its phases, according to Mr. Townsend who has been close to the entire work, the project has succeeded admirably, although this has been gained only through gradual development and improvement.

Accomplishments Summarized.—In the final analysis of cost, it was found that expenditures for the sludge dewatering plant represented about 10% of the total, the other 90% being accounted for by collecting sewers, intercepting devices, and the entire sewage purification part of the plant. Even at its beginning, the operation of this complicated purification plant, much of it entirely new in character, was a success. Subsequently, research and development have permitted further improvement in the process of sewage aeration and plant control so that the anticipated capacity has even been exceeded substantially.

* Asst. to Chf. Engr. and Acting Chf. Engr., Sewerage Comm., Milwaukee, Wis.

Originally designed to accommodate 85 000 000 gal. per day, it has succeeded in treating successfully approximately 90 000 000 gal. per day, yielding, thereby, approximately 100 tons of saleable commercial fertilizer. In all, more than 40 000 tons of fertilizer had been produced up to 1928, at a value of about $15 or more per ton in the market, and a reliable outlet for the available material has been created. Viewed, therefore, in its entirety, this Milwaukee sanitary plant seems to be the most modern, unique, and complete in the world, according to Mr. Townsend. It also holds a remarkable record in the extent of its self-support financially.

Early Experimentation.—In the early days of this work, little was known regarding the applicability of any system of disposal to local needs. As a result, several years of experimentation were carried on. The latter part of this was devoted exclusively to the activated sludge process and eventually this was perfected to the point where construction seemed to promise success. The final completion of the disposal plant occurred during the summer of 1925, after nine years of work. The sludge dewatering plant was placed in operation the following November.

Among the processes tried experimentally were included Imhoff tanks, sprinkling filters, chemical precipitation, screening, chlorination, and others One by one these were eliminated from final consideration. Meanwhile, the activated sludge process came under consideration and shortly the preliminary experimentation showed its alluring possibilities, and led to the complete exclusion of the other processes studied. Although the sewage of Milwaukee has a constitution varying with the locality and time of year, for the purpose of analysis an average condition had to be assumed. This so-called hypothetical sewage contained approximately 250 parts per million of suspended matter.

During the progress of these preliminary tests, the efficacy of diffuser plates over air jets was established. Thereafter, they became standard for tests. At first, also, the tests were confined to the intermittent process—the so-called fill-and-draw method. After continued tests, however, a design for the first continuous-flow activated sludge tank was perfected and actual testing of this was begun in June, 1915.

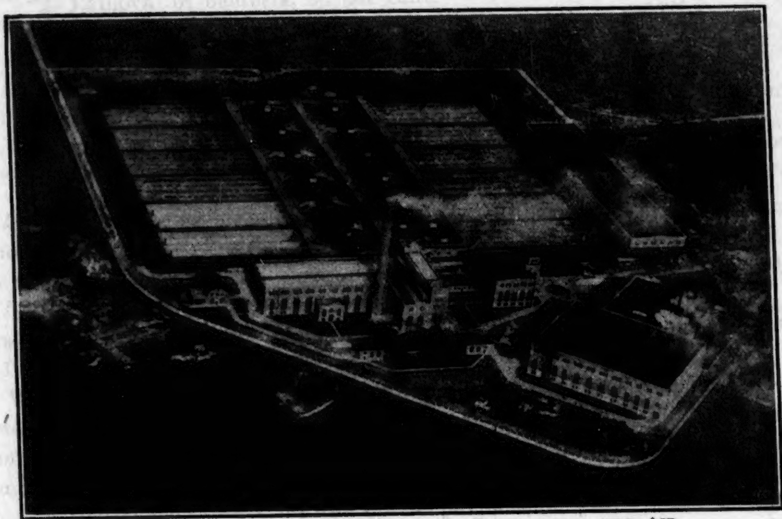
Reason for Activated Sludge Choice.—Thus, by 1916, the value of the method was established, so clearly in fact, that the engineers felt justified in recommending its adoption. This was based on the following grounds:

1.—It gave a higher degree of removal of suspended solids and bacteria than the other methods.

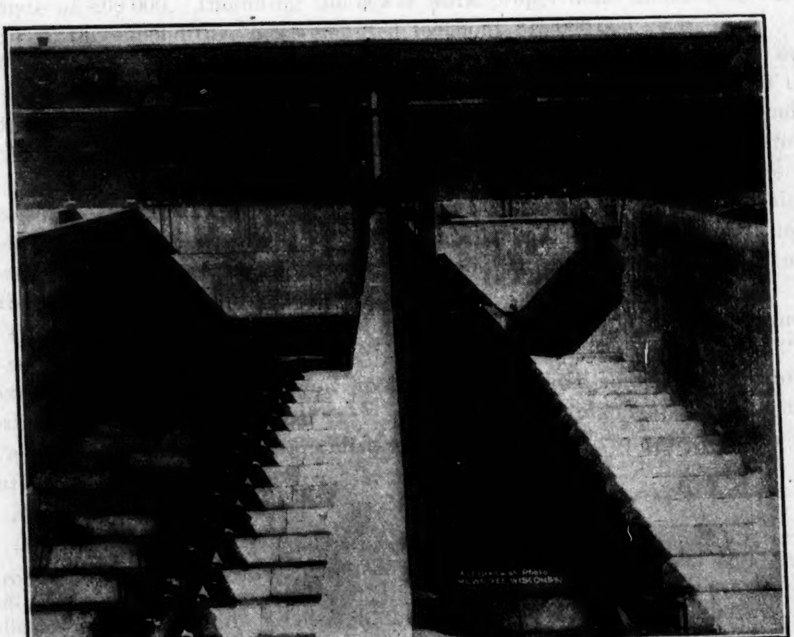
2.—The resulting waste sludge possessed ultimate disposal value and presented an attractive possible return.

3.—The method seemed to be economical of space, this being due to the high returns for treatment obtainable.

Through the continuation of the experimentation, the engineers felt justified to expect the following behavior of a large-scale plant:



THE MILWAUKEE SEWAGE DISPOSAL PLANT FROM THE AIR.



INLET AND OUTLET END OF AERATION TANK, MILWAUKEE SEWAGE DISPOSAL PLANT.

"1. Removal of suspended matter, 95 per cent.
"2. Removal of bacteria, 90 per cent.
"3. Stability of effluent, 72 hours (methylene blue method).
"4. Unit rate of aeration tank treatment, 15 000 000 gal. of sewage per acre per day.
"5. Settling capacity, 1 600 gal. of sewage per sq. ft. of horizontal liquid surface per day.
"6. Satisfactory dewatering and drying of the sewage sludge through the use of drum type vacuum filters and indirect heat dryers."

Data for Capacity.—The site selected for the disposal plant was known as Jones Island, a low-lying district which required extensive fill for its final use. The approximate hydraulic operating elevation was determined as about 11 ft. above lake datum. The entire work, including sewage collecting systems, was predicated on the expected growth of population to about 860 000 in 1950.

Within the city limits itself, the new intercepting sewers had been completed by 1924 at a total expense of $7 500 000. In addition, sanitary work in communities adjacent to Milwaukee was put under the control of a separate commission which worked in conjunction with the Milwaukee Commission. This revised the prospects for the entire purification project, as the contributing population in 1928 totaled 650 000, and the estimated 1950 population had to be correspondingly increased to about 1 000 000 as against the original estimate of 860 000. Including the sewer work proper done under both these headings, the expenditure has amounted to about $15 000 000.

Preliminary Treatment.—In connection with the details of aeration and sedimentation units, Mr. Townsend referred to his previous paper before the Society.* Crude sewage arrives at the plant through inverted siphons under the harbor entrance. It originates from varying elevations, and, therefore, reaches the plant in the corresponding relations—the low-level sewage at Elevation —16 going into two coarse screening compartments, and the high-level sewage at Elevation +11 into a single coarse screening compartment. Thereafter, the low-level sewage (screened) is forced by centrifugal pumps to the upper level for joint treatment with that from the higher source.

Thence, it flows through grit compartments to fine screens which remove all suspended matter of more than $\frac{3}{32}$ in. Flowing farther it has added to it a fixed proportion of previously aerated and concentrated sludge. The required mixing is effected in a channel provided with diffuser plates and leading to a gate-house where the flow is divided, as conditions may require, to north and south aeration tanks.

Route of Sewage.—Quoting from Mr. Townsend's paper:

"The mixed liquor is admitted to each of the aeration tanks in service through 30-in., sluice-gate controlled, inlets and after having passed through the reverse-flow aeration tanks, it then enters 24 by 12-in. Venturi meter tubes, one for each unit. Through this device the rate of flow and quantity is recorded, and the aerated liquor is conveyed to the mixed-liquor supply channels which border each of the octagonal sedimentation tanks on three sides.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 837.

"From these supply channels, which also are equipped with air diffusers, the flow enters each of the sedimentation tanks from two opposite sides through submerged inlets. The clarified effluent is collected and taken off at the liquid surface by surface channels. Thence, it is discharged into two outlet channels connecting directly with the point of effluent discharge into the lake.

"*Handling Sludge.*—The settled sludge is drawn from each tank bottom through a 16-in. center outlet pipe located beneath the tank bottom. This pipe terminates in the form of an adjustable draw-off pipe arm through which the sludge discharges directly into one or the other of duplicate aerated sludge collecting channels located in the center gallery.

"The sludge-collecting channels, each equipped with a sluice-gate control, terminate as open channels near the mixed-liquor inlet gate-house. At this point they converge through a forebay into a single 54-in. concrete conduit, constructed underground and terminating in a sludge pump-well in the power house.

"Through the use of low-lift, slow-speed, centrifugal pumps, the sludge is removed from the sludge pump-well. It is elevated, metered, and returned through a 48-in. cast-iron, force main to the screened sewage. By multiple outlets the sludge is dispersed into and through the raw sewage.

"All the sludge in excess of the quantity necessary for re-circulating purposes, is conveyed through underground cast-iron mains, after having been drawn either directly from the return-sludge main or from the sludge-conditioning sedimentation tanks, to the acidification tanks and filtration-control house adjacent to the sludge-dewatering plant."

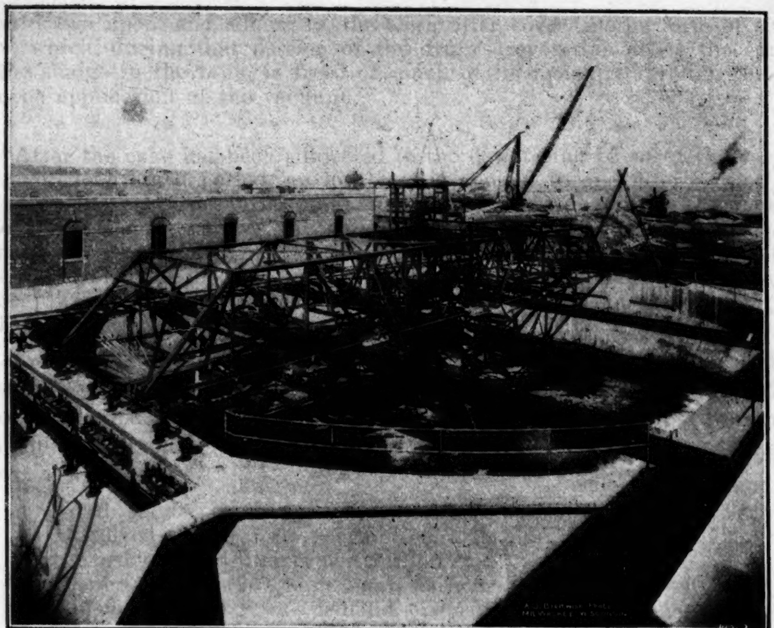
Mechanical Plant.—The prime movers for the entire plant are concentrated in the boiler and power houses. The extent of these mechanical features, as Mr. Townsend mentions, represents a wide departure and hitherto unprecedented necessity in the field of sanitary engineering. The sewage pumps themselves (three in number, including one spare) are each of 30 000 000 gal. per day capacity. A float control is provided to aid in synchronizing the discharge with the inflow.

Then there are three slow-speed centrifugal sludge pumps, used to remove the settled sludge and return it to the raw sewage for re-circulation. These are arranged to work at rates of 6 000 000, 9 000,000, and 12 000 000 gal. per day each. To supply the air for aeration, turbo-blowers are used, driven by steam turbines consisting of four units each with a rated capacity of 30 000 cu. ft. of air per min. They secure a compression of 10 lb. per sq. in. Before the air is compressed, it is filtered. All these pumps and blowers are equipped with Venturi meters for measuring the flow.

Sludge Dewatering and Drying.—One of the large elements of the work is the sludge-dewatering plant. The filtering plant consists of 24 vacuum filters, each 12 ft. in diameter by 14 ft. long. Each filter operates individually. Thereafter, as Mr. Townsend described it:

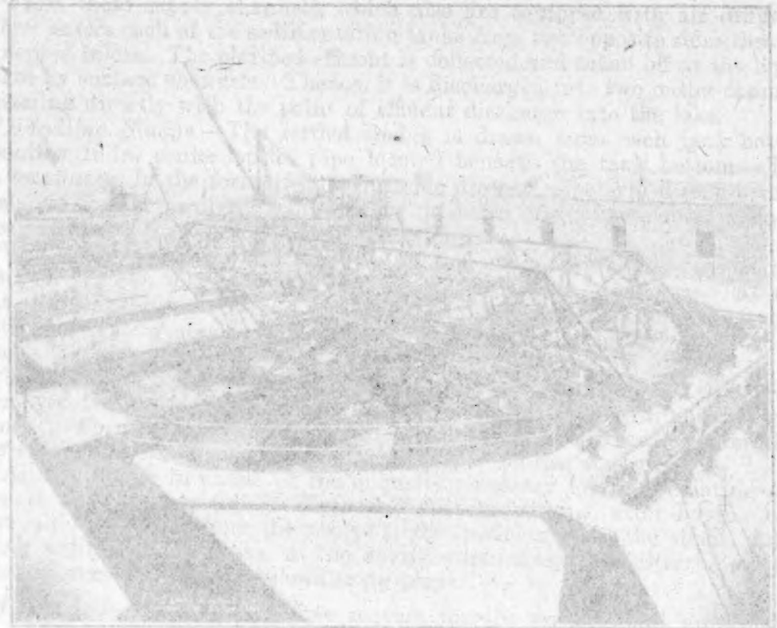
"The waste sludge supplied to the circular bottom tanks, in which the drums revolve, and from which they draw the sludge, is, after having been acidified, pumped from the acidification tanks located near the main building, by motor-driven displacement plunger pumps, into and through a system of piping located below the filter operating floor; the necessary supply being admitted at will, and by hand, into the units it is desired to use.

"Through a complete system of vacuum piping, and through the use of reciprocating vacuum pumps, the solids contained in the sludge are caused

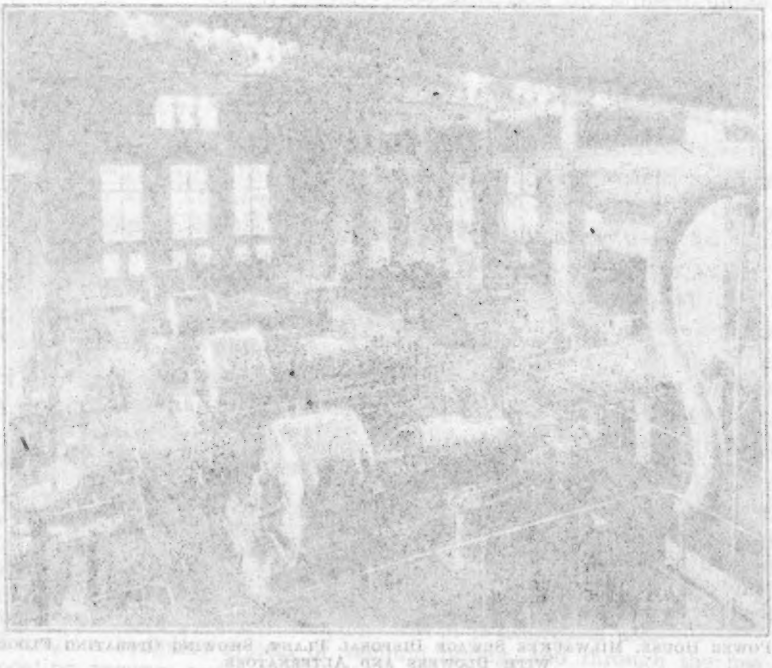


MILWAUKEE SEDIMENTATION TANK, WITH SLUDGE-REMOVING APPARATUS.

POWER HOUSE, MILWAUKEE SEWAGE DISPOSAL PLANT, SHOWING OPERATING FLOOR WITH BLOWERS AND ALTERNATORS.



MILWAUKEE SEDIMENTATION TANK, WITH SLUDGE-REMOVING APPARATUS.



POWER HOUSE, MILWAUKEE SEWAGE DISPOSAL PLANT, SHOWING OPERATING FLOOR, WITH BLOWERS AND ALTERNATOR.

to be drawn upon, and adhere to, the cloth filter cover, in the form of sludge cake, which, during that portion of the drum's revolution above the surface of the sludge in the tank, is freed of much of its moisture through the continuous application of the vacuum.

* * * * * * *

"After the cake has been subjected to the dewatering effect of the vacuum for a predetermined period, an air blow is applied to one filter section just ahead of the cake discharge point, and this blow, together with the use of a scraper extending across the entire face width of the drum, loosens and scrapes the cake from the surface of the filter. From this point, it falls upon belt conveyors, is collected, and deposited upon an inclined belt conveyor of larger dimensions, and conveyed up and into the dryer house along the feed end of the dryers."

Product and Cost.—The final drying is accomplished by direct and indirect heating so that eventually the sludge is dried, pulverized, and bagged for sale. The effect of operation of the plant is disclosed by Mr. Townsend in the following table:

	Fine screen effluent.	Plant effluent.
"Total solids, parts per million..........	1 000	750
Dissolved solids, parts per million........	700	725
Suspended solids, parts per million	250 to 300	25
Bacteria count (26° cent., agar, 48 hours).	3 000 000†	40 000
Bio-chemical oxygen demand, parts per million, (20° cent., 5-day incubation).	200 to 250	0 to 6
Stability, in hours....................	....	120†
Percentage of suspended solids removal...	....	92 to 98
Percentage of bacterial removal.........	....	95 to 98
Nitrogen, total as NH_3.................	35	4.50
Nitrates	....	4.00
Percentage of nitrites..................	....	1"

For an entire year the estimated cost of operation, including payroll, administration, coal, chemicals, supplies, repairs, maintenance, insurance, sales, and miscellaneous expenses, is almost \$900 000. During 1928, the sale of the sludge approximated \$600 000. The unit cost may be found from the average daily sewage flow of 90 000 000 gal., giving about \$9.70 per 1 000 000 gal. of sewage treated, or, in terms of population, about \$0.50 per capita per annum. Mr. Townsend expects that these favorable results will at least continue.

Operating Experience.—Summarizing the operation of the plant, Mr. Townsend states that:

"None, except difficulties of a very minor nature, has been experienced in the general operation of the sewage purification plant, the tanks and all appurtenant equipment having functioned to a great extent as originally anticipated, the work of routine operation having been systematized to the point where the repetition of previously experienced events common to sewage flow characteristics, are anticipated and accordingly provided for.

"The quantity of air used per gallon of sewage treated has, for some months past (1928), been somewhat less than 1 cu. ft., the flow through individual aeration tanks having been increased while the quantity of air per diffuser plate per minute has remained the same—approximately 1 cu. ft.

"Under certain of the higher mixed-liquor solids concentrations, the sedimentation tanks during peak flows have indicated that their capacity has been reached; this condition, however, for the present at least, can be accom-

modated conveniently, inasmuch as the solids content control is subject to the will of the operator.

"From all indications there are ample aeration tanks for a number of years to come, but, before any appreciable additional quantity of sewage is admitted to the plant, additional sedimentation capacity must be provided.

"The control methods which have been developed relative to the filtering of sludge would indicate that there are a sufficient number of filters now installed to accommodate an appreciably increased sewage flow.

"The sludge dryers are operating at about the limit of their capacity, and, in looking forward to increasing the sewage flow into and through the plant, the matter of additional dryer capacity is a very pertinent consideration."

DISCUSSION

MILWAUKEE SANITARY WORK

Value of Heating.—Following Mr. Townsend, several engineers discussed the various phases of the Milwaukee work. Unfortunately, sickness and other hindrances prevented the complete discussion that was expected.

It was brought out by W. R. Copeland, Affiliate, Am. Soc. C. E., that one of the handicaps in operation is caused by the difficulties of treatment during cold weather. In other words, the biological organisms essential to the digestion of the sludge do not find the conditions propitious during the winter. To improve matters, chemical means have been used at Milwaukee to supplement the natural warmth of the material, and it is possible to handle the sludge at all seasons. Otherwise the twenty-four filters provided would have been insufficient.

An important economy consists in the utilization of the gas generated by the reactions, which, by combustion, transfers its heat to a circulating water system and thence to the screenings. The saving in time amounts to as much as two-thirds, that is, a reduction in the period of fermentation from about 120 days under the normal winter temperature, to about 40 days, due to the gas heating. Another practical consideration is the resulting ease of disposing of the material, both in cold as well as in warm weather.

As a result of the research carried out, Mr. Copeland concluded:

"It has been possible to place at the disposal of the citizens of Milwaukee a method of disposing of their sludge at all seasons of the year; and a method of biological treatment of the screenings, which again will reduce that very serious problem to its least common denominator. In the construction of great sewage disposal plants, that bugbear of the sanitary engineer, the disposal of the sludge and the screenings, has become a matter of the past."

Impressive Economies.—According to T. Chalkley Hatton, M. Am. Soc. C. E., the Milwaukee plant is unique in this matter of disposing of the sludge at a commercial profit, in addition to its treatment of sewage to a high sanitary standard. He quoted the commercial value of the sludge as about $17 per ton at the plant. Further, he anticipated that reductions in cost of clarifying the sludge would be obtained. As regards further economies, he mentioned that at the beginning the estimate was 1½ cu. ft. of air per gal. for

sewage treated. In practice, it has been possible to reduce this to as low as 1 ft.; and further research is being carried on by those who hope that this can be cut still further to perhaps 0.8 ft. per gal.

As the largest item of cost in clarifying a sewage is in the compression of air, these improvements point to greatly increased economy. This is especially emphasized by comparison of costs of purification in other plants which amount to three to six times as much. As a general proposition, he claimed that the activated sludge process cannot be a complete success without the commercial disposal of the sludge. This, of course, is almost impossible in a small plant; "but, on the whole, the larger installations cannot be successful without going to the limit and getting rid of this awful nuisance of sludge."

Experiences in Chicago.—In a prepared discussion from Langdon Pearse, M. Am. Soc. C. E., read by Samuel A. Greeley, M. Am. Soc. C. E., Mr. Pearse did not fully anticipate the same optimistic results of the sludge treatment that were presented by Mr. Townsend. He explained that, in the work of the Chicago Sanitary District, it did not seem feasible to follow the Milwaukee process. To indicate more fully the methods used in the Chicago North Side work, Mr. Greeley showed a number of slides indicating details of the various parts of this interesting development. One of the distinctive features, he explained, was the "spiral aeration tank having an incline bevel at the top and aeration plates in the side."

SANITARY OR PUBLIC HEALTH ENGINEER?

By C. M. Spofford,* M. Am. Soc. C. E.

One of the most interesting features of the meeting came out of the ensuing discussion on the question "Is Public Health Engineer a Better Title Than Sanitary Engineer?" The topic was presented in a general way by C. M. Spofford, M. Am. Soc. C. E., who referred to the definite suggestion that the name of the Sanitary Engineering Course at the Massachusetts Institute of Technology be changed officially to "Public Health Engineering". Actually, the American Public Health Association has changed the title of its Sanitary Engineering Section to Public Health Engineering Section.

He emphasized the confusion also in the public mind between the Sanitary Engineer and the plumber. Even some men of considerable influence do not have a clear conception. The American Society of Sanitary Engineers is nothing more than an association of plumbers. The matter resolves itself into this—which of the terms more completely describes the present engineering field? Professor Spofford's attitude was that there is no prescriptive right to the use of a title, and plumbers could easily become nominally "Public Health Engineers" if that were to their interest.

As long as the public does not understand fully what a Civil Engineer is, it is hardly to be hoped that they will comprehend any other similar title.

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On the other hand, Professor Spofford considered that at least among engineers themselves the term, Sanitary Engineer, had gained a very definite meaning.

DISCUSSION

PUBLIC HEALTH ENGINEER

Varying Views.—This idea found acceptance from others. The historical advantage of the present term appealed to Abel Wolman, M. Am. Soc. C. E., as giving considerable prestige. He expressed the view that the field should be made clear, in so far as the Civil Engineer is connected with it, emphasizing the biological attitude. In short, his interest was concerned with technique and with subject matter, rather than with men. He expressed the fear that the word, "health", would immediately suggest a physician and the addition of the title, engineer, would only confuse the public even more.

Stating frankly that he was trying to view the question as a non-engineer, C. G. Wigley, M. Am. Soc. C. E., urged careful consideration of the change as possessing real merit. Actually, he said the work of the Sanitary Engineer in the minds of the public and of officials is premised on the factor of public health. Present trends seem to favor the later title as widened by the recent establishment of the college degree, "Doctor of Public Health". This title is a happy one, fixing at once the field of work and the professional status in that field. Practicing engineers, however, are beset by difficulties on either hand. A plumber may be glad to take over certain sanitary work, or, on the other extreme, a medical doctor might be called. Thus, the Health Officials are compelled to require only a qualified source for a report—that is, a Sanitary Engineer.

A middle ground was preferred by Thorndike Saville, M. Am. Soc. C. E. He emphasized the need of specialized training, but believed that the title did not matter, one way or the other. The public generally understands the term, Civil Engineer, and if sanitary experience is demanded, it will be able to gauge the man apart from his designation.

"Municipal Doctor".—According to L. L. Tribus, M. Am. Soc. C. E., the chief need of a satisfactory title was in Court proceedings. Is an expert a Civil, Hydraulic, or Sanitary Engineer? He said the term most easily understood by the Court was the characterization as "Municipal Doctor".

In closing the meeting, Chairman Weston mentioned the sources of training as being either up through the various phases of engineering work or through the laboratory. Hence, he thought, it was natural that opinions also should vary. As "sanitary" really is derived from considerations of "health", the two terms are alike and, therefore, the change would not improve the fundamental conceptions.

NEW JERSEY SEWAGE EXPERIMENTS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION

In presenting the following report the Committee wishes to acknowledge the courtesy and co-operation tendered it by Dr. Willem Rudolfs, Chief of the Department of Sewage Disposal of the New Jersey Agricultural Experiment Station.

Work of the Station.—Since the last report* the work of the Station has been directed to developing the fundamental principles underlying:

(*a*) Production of odors and their control.
(*b*) Digestion of suspended solids contained by sanitary sewage in the form of fats, screenings, and sludge.
(*c*) Control of foul odors, "unloading of solids", and sterilization of the effluent from sprinkling filters.
(*d*) Research connected with conditions which control numbers and types of bacteria prevailing at different stages of digestion.

Some of the important results achieved may be outlined briefly as follows:

Odors: Their Cause and Control.—Where the organic matters contained by sanitary sewage decompose in the absence of oxygen, as in a septic tank, bad odors develop. Tests show that more than 90% of the bad odor is due to hydrogen sulfide (H_2S).

The best method of preventing its formation is to maintain the optimum reaction—pH 7.3 to pH 7.6—in the sludge while digesting. If the reaction gets out of balance an application of hydrate of lime in sufficient quantity to bring the reaction back to 7.3 will check the formation of H_2S and destroy the odor.

Studies on the reduction of bio-chemical oxygen demand, the effect of pre-chlorination on cleaning filters, and studies of the effect of chlorine on solids digestion, together with a long series of studies on odor control, have shown that odors emanating from sewage disposal plants can be controlled effectively by the use of chlorine when applied during certain hours of the day during the summer months.

Reaction for Determining pH.—As a result of painstaking research, the New Jersey laboratory finds that the reaction of sludge can be determined readily by any intelligent person if they have suitable equipment. The LaMotte Chemical Products Company, of Baltimore, Md., has placed a "field" testing outfit on the market, based on the laboratory's recommendations. The procedure to be followed in making the test consists of diluting a known volume of sludge with a known volume of distilled water; of adding a measured volume of concentrated color solution; and of comparing the mixture with a series of color standards.

Digestion of Sludge and Screenings.—Foul odors do not develop in tanks containing a large enough volume of well ripened sludge to digest the fresh solids that settle out of the sewage. Experiments have shown that in order to insure good digestion the freshly precipitated solids should be mixed with

* *Proceedings*, Am. Soc. C. E., April, 1927, Society Affairs, p. 185.

previously digested material in the ratio of 10% of raw sludge to 90% of ripe sludge; or, to express it in another way, the "dry weight" of freshly settled solids should not exceed 2% of the weight of dry solids in the ripened sludge.

Analyses of many samples of settled sludge and fine screenings show that, in general, the precipitated materials contain 30% of carbon and 4% of nitrogen, together with various quantities of fats, water, etc.

As digestion proceeds the carbon is first attacked, then the nitrogen, and, finally, the carbohydrates. The carbon is converted for the most part into CH_4, the nitrogen into ammonia (or its salts), and the carbohydrates into fatty acid.

If a long, narrow tank, say, 20 ft. deep, is pictured, with sanitary sewage entering at one end, it is easy to imagine that a pile of coarse paper, rags, garbage, etc., will drop down forming a heap of matter rich in cellulose. The semi-colloidal matters of less specific gravity pass along, settling more slowly and gathering near the middle of the tank. Soaps, fats, and similar carbohydrates pass still farther on and may not settle until after they reach the middle. Meantime, fermentation has started in the pile of cellulose, and carbon dioxide (CO_2), together with other compounds of an acid nature, begins to form. Because they are acid, they reduce the pH of the mass and surrounding liquid. If allowed to continue unchecked, the acidity will become more and more pronounced. The pH may drop from 7.6 to 6.5, 6.0, or even to 5.5.

Unless powdered hydrate, or carbonate, of lime is added to bring the pH back to 7.4±, foul-smelling H_2S and other gases begin to escape, and the tank will foam. As a corrective agent 1 lb. of hydrate of lime (CaO_2H_2) will do as well as 8 lb. of lime carbonate ($CaCO_3$). Any lime added tends to increase the sludge, but the carbonates increase the volume of sludge much faster. Tests show that when treating ordinary sanitary sewage, 4 or 5 lb. of hydrate of lime added once a day will restore in a short time the reaction to neutral (pH of 7.0); then the foaming will decrease and the foul odors disappear.

If the tank has been in service for a long time and if the sludge is in fair condition, the lime m[illegible] simply be spread over the surface of the sludge. It should not be added to the stream of inflowing sewage because, under such conditions, a large part will pass off in the effluent. Again, if it is fed to sprinkling filters, the lime settles on the stones. When, however, a tank is being put into service, the lime should be mixed thoroughly through all parts of the sludge.

Particular attention is called to the fact that if the settled sludge once becomes well ripened, and if the volume of fresh sludge is kept down to the ratio noted, a very few doses of lime will suffice. Care must be taken, however, not to add too much lime, for an excess seems to accelerate the growth of bacteria that produce gas, so that an overdose of lime may cause foaming.

Tests indicate that bacteria growing at pH of 6.8 to 7.2 liquefy the sludge without producing much gas, and bacteria growing at pH of 7.5 to 7.8 produce great volumes of gas.

Lime hydrate is better than sodium hydrate as a re-agent for conditioning sludge. The reason is that besides correcting free acids and raising the pH, lime coagulates colloidal organic matter much better than sodium hydrate.

Nitrogenous compounds decompose with the formation of ammonia (NH_4OH). Apparently, the nitrogen does not decompose as fast as the carbonaceous material. Therefore, analyses of sludges which have been partly digested, often show that the carbon remaining has been reduced so that it is about equal to the nitrogen. This accounts for the fact that sludge, while digesting, decreases to a volume only one-quarter as large as its original bulk.

Optimum Temperature for Sludge Digestion.—Many analyses have shown that the annual temperature of sewage east of the Mississippi, and south of the Canadian line to the Virginias, averages from 51 to 55° Fahr. The normal temperature of sludge in tanks fed by sewage is also 51 to 55° Fahr.; that is, very delicate thermometers fail to register any increase in temperature due to fermentation of sludge in the liquid condition in which it exists in the tank.

As sludge having a temperature of only 55° Fahr. requires from 100 to 120 days to digest, it is important to note that the period of digestion will decrease if the temperaure is raised.

At 55° Fahr., the sludge digests in 120 days; at 72° Fahr., in 40 days; and at 82° Fahr., in 30 days. The great advantage derived from heating the sludge to 72° Fahr. is apparent.

Volume of Gas Formed During Sludge Digestion.—If one throws a lighted match on a bubble of the gas escaping from digesting sludge, the gas catches fire and burns. Many plants are now equipped with roofs and gasometers for collecting the gas. If allowed to digest for a very long time 0.6 cu. ft. of gas can be obtained per capita per day; but a rational figure for average digestion is 0.4 cu. ft.

This volume, if burned under a boiler, connected by a series of pipe coils arranged around the side walls, will give sufficient heat to raise the temperature of the sludge from 55 to 70° Fahr. in winter. The effect of applying the heat to summer sludge would be to raise it to a little more than 82° Fahr., which means that the sludge will digest in summer at maximum speed.

Stirring Advantageous.—Comparative tests made in tanks with and without stirring show that a moderate amount of stirring promotes digestion, helps to maintain the favorable reactions, and sets free the entrained gases at a uniform rate. It also mixes any corrective chemical into the mass of the sludge and tends to flatten out any mounds of sludge that may accumulate.

A question has been raised as to how old the digested sludge may be and still make good "seed". Records show that sludge which has been ripened properly is good for a year at least, but that sludge 2 to 3 years old is hardly of any use.

Engineers have wondered about freezing. Recent tests have shown that it does not hurt sludge to freeze it once, but probably frequent freezing and thawing would eventually destroy the bacteria.

Well-digested sludge contains about 97% of water. If this sludge is allowed to stand, and if the top water is siphoned off, the moisture can often be re-

duced to about 85 per cent. At such a moisture the sludge will still work well for seed; but if the moisture is decreased to 75%, or less, the mass becomes so dry and stiff that it does not mix well and, therefore, does not serve well for seed.

Screenings Digestion.—Screenings obtained by running sewage through plates with small openings can be digested in tanks by the same methods, and under the same controlling conditions, as sludge.

Orange peels, corn husks, melon rinds, etc., will also digest in tanks, but they upset the normal tank reactions, create foaming, etc. For this reason it is not advisable to dump garbage into the public sewers leading to a disposal tank.

Sodium Chloride.—Sodium chloride can be added to sewage entering a sludge digestion tank up to a volume of 500 parts per million, without interfering with sludge digestion. As ordinary sewage rarely contains more than 100 parts of salt per million, it follows that sea water can be run into sludge digestion tanks at a volume equal to the sewage without injury.

Types of Bacteria in Sludge.—As a result of a great many tests for numbers and species of bacteria found in digesting sludge, two main conclusions may be drawn:

First.—The whole numbers found in raw sewage are not a criterion of the number of specific physiological groups present that are active in digesting sludge.

Second.—If the whole numbers of bacteria in the sludge vary from 20 000 000 to 40 000 000 per cu. cm., the sludge is probably well digested and ripe, but where the numbers amount to 100 000 000, or more, per cu. cm., the sludge is not ripe and will not digest to best advantage.

Deductions from the Experimental Work.—A close survey of the facts listed indicates clearly the fundamental importance of the data secured in recent years on the design and operation of sludge and screenings digestion tanks. The following conclusions are evident:

1.—Efficient methods for testing the pH of sludges provide ways of learning "ahead of time", whether or not the tanks are working well.

2.—By controlling the reaction of digestion, foul odors will be suppressed and foaming eliminated.

3.—By heating the sludge with gas generated by the digesting sludge, the period of digestion can be reduced from 2½ to 4 times. This means that smaller tanks can be used, or three to four times as much sewage can be treated per tank.

4.—By the application of moderate quantities of liquid chlorine to the effluent, the growth of flies can be reduced and the effluent from sprinkling filters can be sterilized.

5.—Finally, if the pH and sludge digestion are properly controlled in the digestion tanks, most of the organic matter that causes "unloading" of sprinkling filters will be prevented.

GEORGE T. HAMMOND, *Chairman*,

January 11, 1929. WILLIAM R. COPELAND.

STRUCTURAL DIVISION

JANUARY 17, 1929

Morning Session, 10:00 A. M. to 1:00 P. M.

EXPERIMENTAL INVESTIGATION OF CONCRETE ARCHES

By W. M. Wilson,* M. Am. Soc. C. E.

The first talk of the session was to have been delivered by Clyde T. Morris, M. Am. Soc. C. E., covering the report of the Special Committee on Concrete and Reinforced Concrete Arches. As Professor Morris had given a résumé of this work at the previous general session of the Society, he deferred in favor of W. M. Wilson, M. Am. Soc. C. E., who was to speak on "Experimental Investigation of Concrete Arches". Professor Wilson, as a member of the Committee, has been conducting large scale tests.

The test structures themselves could hardly be called models, Professor Morris commented, as they were 18 ft. long and built identically with similar structures for service. The most important features of Professor Wilson's work have already been extracted and printed with the Special Committee report.

DISCUSSION

EXPERIMENTS ON CONCRETE ARCHES

By George E. Beggs,† M. Am. Soc. C. E.

Celluloid Models.—In the interest of verifying certain results from the larger structures, George E. Beggs, M. Am. Soc. C. E., had made tests on celluloid models of 2 or 3-ft. span. Some of these models he had on exhibition, for demonstration to interested members. His discussion, however, was confined mostly to the tabulated results obtained from his tests.

According to Professor Beggs, tests have been made, even previous to 1923, to determine the strengthening effect of the superstructure on the arch rib itself. These established the fact that apparently the effect of this co-operation was quite marked. Using this knowledge to advantage, French engineers later constructed the first bridge in which an attempt was made to

* Research Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

† Associate Prof., Civ. Eng., Princeton Univ., Princeton, N. J.

save the material. Stresses in this latter bridge were checked by experiments on plate-glass models about 18 in. in span, analyzed by polarized light. In this bridge flexural hinges were formed by reducing the rib of the bridge to one-quarter its normal size, through which narrowed area the main reinforcement of the soffit and back were crossed, giving resistance to shear and thrust, but not to moment.

Specific results were furnished by Professor Beggs covering tests at Princeton University, which demonstrated that bending moments in the rib were greatly reduced by reason of the continuous superstructure. Influence lines showed graphically the reductions of bending moment which are especially significant since these stresses are the largest single factor in determining the strength of the bridge. The diagrams showed influence lines for rib stress both with and without the decking.

Checking Effects of Decking.—Still another feature emphasized by the test was that the concrete deck actually did not behave as ordinarily assumed. The deck is usually computed as a continuous beam on rigid supports, or at least this is the basis for design. Actually, however, the supports are far from rigid inasmuch as they must yield with the deflection of the arch itself under load. Hence, the deck may have stresses induced due to the distortion of the rib. Still another interesting effect of this action is the tension or compression in the floor due to the distortions, causing stresses which are ordinarily neglected entirely.

Other interesting tests explained were those on celluloid models of the reinforced concrete arches tested by Professor Wilson. Besides the deflections due to loads, readings were taken to determine the effects of temperature changes. The models were constructed so that the abutments could be moved simulating the conditions of stress produced by temperature change. Influence lines were derived for thrust, shear, and moment in the arch and in the floor. In some cases, expansion joints were introduced; and their influence on the stresses in the arch and superstructure was studied. In a general way, the comparison between test results from the larger concrete arch and the small celluloid model was quite close. Professor Beggs pointed out in closing that the advantage of the integral superstructure was not nearly so noticeable under symmetrical loads as when the loads were not symmetrically placed. In the latter case the action of the deck in reducing stresses in the arch rib was marked.

ARCH TESTS

By A. Burton Cohen,* M. Am. Soc. C. E.

A study of Professor Wilson's results showed A. Burton Cohen, M. Am. Soc. C. E., that:

"The tests prove the validity of the elastic theory in several convincing ways. They indicate that the strength of an arch having a ratio of unsup-

* Cons. Engr., New York, N. Y.

ported length to width equal to 30 is not weakened by a tendency to buckle. The ratio of slenderness is an important consideration because the designer is often confronted with the choice between a braced and an unbraced rib."

Expansion Joints and Cracks.—Regarding expansion joints, Mr. Cohen claimed that for spans up to 100 ft. they might be omitted, but that for spans of about 200 ft., possibly four joints might be required, "because of the comparatively large moment that the solid floor would have to resist in overcoming the effect of the rise and fall of the arch ring". Noting Professor Wilson's suggestion that under known conditions it might be possible to locate expansion joints where they would not be detrimental, Mr. Cohen suggested, from the reverse angle, that,

"The position of the expansion joints being fixed, the arch ring and floor construction may be so proportioned or reinforced that considerable economic and structural value may be gained by the combined effect".

Applying some of the thoughts of the test to his experience with various arches on the Delaware, Lackawanna and Western Railroad, Mr. Cohen showed several slides. He remarked that in the earlier days it was thought that much of the movement of the arch was due to the early shrinkage of the concrete; hence the desire to build the rib in separate blocks which were to be joined with concrete after the original shrinkage of setting had taken place. The engineers went still further and struck the arch centers, putting the ribs into action before the floor was laid. After noticing that serious cracks occurred under loading, expansion joints were introduced into the later structures. Had it been realized at the time how much benefit integral construction would have given, a considerable saving would have been possible.

GENERAL DISCUSSION

ARCH TESTS

Effect of Varying Values of EI.—Various computations were made, it was revealed by Hardy Cross, M. Am. Soc. C. E., in connection with Professor Wilson's tests. The results were quite interesting. To quote Professor Cross:

"On a certain section we made computations for about 150 different combinations or variations in the values of $E I$ in order to find out what possible effect these variations might have on the moments which would exist according to the elastic theory. * * * The variations produced very little effect on the moments which were computed.

"In other words, to a certain extent the application of a theory of elasticity and the analysis of concrete arches is a problem in probability, but it is a problem in probability in which we are reasonably sure on the basis of this investigation that the variations from the results which are secured by the ordinary elastic analysis, assuming a constant E, will not result in any very radical departure from what might be expected."

Expansion Joints.—In a prepared discussion read by Professor Beggs, Henry C. Tammen, M. Am. Soc. C. E., dealt partly with the question of joints. He stated:

"Frequently the question of whether to provide articulation points or expansion joints at points intermediate between the piers is a difficult one to settle.

"If the joints are omitted there is a greater or less participation of the deck structure in the arch stresses and just to that extent the problem is made indeterminate. For as a result of this deck action it is not possible [by ordinary analysis] to evaluate the stresses either in the deck members or in the arch ribs. As far as dead load stresses in the arch rib are concerned the construction program is usually such that there is little deck participation except for dead load of pavement and hand-rails. As far as live load stresses in the arch rib are concerned, it would seem evident that the stresses are reduced as a result of the deck participation unless the ratio of rise to rib thickness is unusually small. The author's measured results confirm this. As far as temperature stresses are concerned, it would appear that they might be reduced near the crown and no doubt will be increased near the springing. Where the ratio of rise to rib thickness is small, both temperature stresses and arch shortening effects are large and the increased thrusts and increased temperature moments near the springing as a result of omitting joints in the deck might well prove serious. Accurate methods of determining such stresses are therefore needed.

"Considering the stresses in the deck members there is the same degree of indetermination. Yet the deck must be designed both for vertical loads and for the indeterminate effect of participation in the arch action as regards applied loads and temperature changes."

It seemed possible to Mr. Tammen that if in the tests the load had been placed within the deck spans instead of at column points, the corresponding heavy local bending stresses might have resulted much less favorably. He considered expansion joints as valuable in making the structure determinate, which action was strikingly proved in Professor Wilson's tests. Admitting that for many reasons expansion joints are undesirable, Mr. Tammen felt that the results of the tests were not of complete value until the difficulty of eliminating the joints was cleared up.

Initial Stresses.—In the ensuing discussion, T. K. A. Hendrick, Assoc. M. Am. Soc. C. E., mentioned the effect of friction in the test apparatus. Another matter that appealed especially to H. H. Quimby, M. Am. Soc. C. E., was the question of initial stress that is almost inevitable in a concrete arch. This he said was due partly to shrinkage but perhaps in largest measure to the deformation of the supports. The best method he could suggest of avoiding this was to embed in the arch a structural rib capable of carrying the entire dead weight of a structure.

He claimed that the tests themselves had shown that the maximum bending stress was at the springing line and he felt that this would be increased if anything by the deformation of the centering. As one objection to tests, he mentioned the difficulty of maintaining a proper dead weight of the structure so that the stresses would be comparable to that of the actual unloaded arch.

Required Computations.—It was brought out by B. F. Hastings, M. Am. Soc. C. E., that the tests showed inconsistency between variable stresses and secant moduli at some points, to which Professor Wilson replied that it was best not to "accept the secant modulus as a measure of strength but as some indication of it." Professor Hastings went on to state that he felt the best proof of the value of the elastic theory for handling indeterminate structures was in the success of the construction in service.

He felt that highly theoretical computations were quite beside the mark and that some "optimistic handling of figures" is required in order to check the results of tests with theory. It is true the elastic theory is the only basis of design, but he had "a general feeling that hair-splitting as applied to various types of structures has its limits".

Variation in Values of I Unimportant.—One phase of the report that appealed to F. E. Richart, Assoc. M. Am. Soc. C. E., was the conclusion that a considerable variation in the moment of inertia along an arch rib has no great effect on the reactions. He believed that this idea could be carried over into other forms of continuous members, such as frames or beams. Professor Richart noted that although frequently failure had occurred at sections not under maximum stress, nevertheless, in nearly all cases, the points of failure were subject almost entirely to direct stress with very little flexure, which, he said, "is in accord with what we know of the failure of members in flexure where the strain fiber stress may be considerably higher than the cylinder strength."

In response to question by F. H. Constant, M. Am. Soc. C. E., as to the value of E to be used in computing stresses, Professor Wilson explained that theoretical stresses were based upon a value of E of 2 500 000 lb. per sq. in., whereas the measured value of the stress was based upon the curve of average-stress mean-strain for that particular section. The linear distribution of stress in a given section is assumed by the value of E varied according to the load and the section considered.

Expansion Joints to Justify Themselves.—As a result of all the discussion that had taken place, Chairman J. J. Yates, called upon the author to comment on the questions raised. First, Professor Wilson mentioned the question of expansion joints. He did not claim that expansion joints intermediate between the piers should not be used. Tests on the Danville, Ill., Bridge showed that joints adjacent to the crown could be omitted without detriment. The current series of tests indicate that expansion joints weaken the structure, but this does not mean that they should not be used somewhat near the center of the span—it all depends on the relative dimensions of the arch. In any case, an expansion joint must justify itself.

Taking a specific example, he discussed in detail the effect of expansion as measured on the Danville Bridge. Certain expansion joints did not operate at all, while others acted as expected. To explain why this may have been true, he showed that the motion of the joint was compounded by changes in the arch and in the deck; as the arch rose through expansion, the spandrel columns rotated and similarly the deck lengthened. It just happened, Professor Wilson claimed, that in this structure the geometrical relations were such that the two movements compensated. What would have occurred for different dimensions is another matter; hence, his strong feeling that expansion joints should justify themselves in every particular case.

Investigation Requires Final Analysis.—In the matter of hair-splitting design, he could not agree that the mere structural integrity of any arch was proof of its accurate design. In some cases it might be ten times too strong and in others just barely able to stand. He went on,

"To take the fact that a structure stands up as proof that it has been satisfactorily designed, is a fallacy. The investigator is, of necessity, compelled to make finer distinctions than a designer. He should try not only to determine what happened, but what particular factor caused it to happen. Minor variations that are of no interest whatsoever to the designing engineer are of tremendous interest to the investigator."

Afternoon Session 2:00 to 4:30 P. M.

YADKIN RIVER BRIDGE TESTS

By Clyde T. Morris,* M. Am. Soc. C. E.

As official representative of the Society on a Joint Committee to make extensive tests on a modern arch bridge, Clyde T. Morris, M. Am. Soc. C. E., had an excellent opportunity to study important questions in arch design and construction. He gave the gist of his more complete report, illustrating the important features by diagrams and photographs.

Testing a Full-Sized Bridge.—The bridge available for these tests was only about 5 years old at the time the experiments were undertaken, and consisted of main arch spans, 158 ft. long. Due to changes in the hydraulic use of the Yadkin River, this particular structure was to be inundated in order to impound water for hydraulic development. A higher level bridge was to be built, and the old one destroyed. This situation presented an unprecedented opportunity for large scale tests which was eagerly seized by engineers.

Primarily, the investigation was under the joint auspices of the North Carolina Highway Commission and the U. S. Bureau of Public Roads. The time normally available was so short that the Yadkin River Bridge was closed prematurely for these tests, a temporary ferry being installed. In the interest of a well-rounded investigation, representatives were invited from a number of organizations, including universities, Government Bureaus, State Departments and National Societies.

Aims Sought.—The objectives of these tests according to Professor Morris, were as follows:

"(*a*) To compare the measured deformations of a full-sized, reinforced concrete arch rib with the deformations as determined by the theory of elastic structures, when the rib is as free as practicable from the restraining influences of the superstructure, and loaded to produce stresses of moderate intensities.

"(*b*) To make the same comparisons when the rib is loaded to produce stresses of high intensities.

"(*c*) To determine the effect of the superstructure on the deformations of the rib, by comparing measured deformations when the superstructure was intact, with measured and computed deformations of the free rib; that is, the rib free from restraint by the superstructure.

* Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.

"(*d*) To compare the measured deformations of the rib, both with and without the restraining action of the superstructure, with the deformations as determined from analyses made by the use of an elastic model."

The Structure.—Because of the advantages of the arch type of bridge and the large annual expenditure in the construction, it was felt that the labor of establishing principles of economical arch design without sacrifice of safety would greatly repay the cost. The great value of testing an actual structure was in locating the effect of departures from the ideal conditions ordinarily investigated in the laboratory.

The middle one of the three arches was chosen for tests. It consisted of two ribs about 11 ft. on centers, having a clear span of 146 ft. 3 in. and a rise of 28 ft. 3 in. Toward the abutments the roadway is carried on columns, but in the center of the span it is integral with the rib, expansion joints being located at the ends, quarter-points, and middle.

"Observations of these sliding joints at the quarter-points and middle, before the tests were started, and during the tests with the deck intact as when in service (Series 1), showed no evidence that any motion had ever taken place. A thin coating of mortar over the edges of many of the plates showed no cracking."

Program of Weighting.—To apply the load two wooden tanks were used, each 12 ft. 6 in. by 20 ft. in horizontal section and 18 ft. high inside. They were equipped with rollers so that when empty they could be easily moved. The procedure was to locate the tanks in the desired position, transfer the load by jacks to the proper points, and pump in the desired quantity of water. Observations were taken for the following loads: Empty tank, 47 000 lb.; the tank with increments of 91 000, 182 000, and 273 000 lb. of water load. Through the spandrel columns these loads could be transferred directly to the ribs, the lightest load corresponding to the effect of four 14-ton trucks moving in two lines across the bridge.

Sequence of Tests.—The procedure for the test was explained by Professor Morris, as follows:

"Series 1 of the tests was run with the superstructure intact as when in service. Only one tank was used, in order to keep the maximum unit stresses within moderate limits, and this was moved forward one panel at a time, from the end to the center of the span. The loads were thus applied at each successive pair of panel points.

"Before Series 2 and 3 were run, the continuity of the superstructure was destroyed by cutting through the floor-slab, girders, and hand-railing over each cross-beam. The ends of the girders were then jacked up and new bearing-plates inserted at each girder bearing. These bearing-plates had planed surfaces and were thoroughly greased. The effort was made to place the structure as nearly as possible in the condition of unrestrained ribs.

"For Series 2 the same loads were repeated as in Series 1.

"For Series 3 two tanks were placed in such position as to produce maximum stresses in the rib with the intention of producing failure if possible.

"*Theoretical Calculations.*—Very careful field measurements were made of the actual dimensions of the structure and these were used in making all calculations. The theoretical shears, moments, and direct stresses were

calculated at each section where telemeters were placed, assuming that the rib was not restrained by the superstructure. The rib was regarded as having fixed ends."

Model Analyses.—Using a celluloid model, including the three arch spans of the bridge, the following deformeter analyses were made:

"(*a*) With the deck cut at the expansion joints, but still integral with the tops of the spandrel colmns.

"(*b*) With the deck cut loose from the tops of the spandrel columns, but reconnected to them by welding small flexible webs of celluloid across the cuts, in order to simulate as nearly as possible the conditions in the field.

"(*c*) With the superstructure entirely removed from the rib. This analysis, of course, checked the mathematical analysis very closely."

Deflections and Shears.—Complete field readings were taken to observe the various actions of the bridge. The horizontal movement of piers was measured by means of wires stretched along the faces of the arch. In addition, pier rotations were measured by clinometers or level bars. During the entire six months of observation, Professor Morris notes "no measurable movement of the pier was observed."

Temperature likewise was measured at various points, which when compared with the movements of the crown showed that the average relationship was $\frac{1}{40}$ in. for each degree centigrade. In this feature also theory and observations agreed. Vertical deflections were measured from horizontal wires under constant observation. As would be expected, the actual deflections were less in Series 1 than those calculated because of the restraining influence of the spandrel columns and deck. The reverse was true in Series 2, where the agreement was close.

Behavior of Materials.—Still another important series of experiments involved deformation or strain, as applied to the north rib only, which had the benefit of shade. Electric telemeters gave the results for the concrete, both extrados and intrados, at nine sections of the rib. Similarly, stress changes in the steel were measured at the springing line and at the crown. In these instances, also, the strengthening effect of the concrete-deck structure was noticeable. Under the water loading the measured compression stress amounted to as much as 1 600 lb. per sq. in., to which should be added that due to the dead weight, including the tank, so that actually the stress probably reached 3 000 lb. per sq. in. Even these high stresses did not apparently affect the general agreement between computed and measured stress.

Comparisons of the steel stresses yield interesting results. The assumption that the concrete takes no tension gave stresses in general larger than the observed stresses and this was especially true of the high tensile stresses observed. On the other hand the actual stresses were found to be only slightly greater than those based on the assumption that the concrete takes the proportional part of the tension.

It was found that a maximum load placed at the center of the adjacent span produced negligible deformations. Concrete cylinders, according to Professor Morris, were cut from the curtain-wall and from the arch rib after the

other tests; these gave values for the modulus of elasticity of 4 500 000 and 3 930 000 lb., respectively; and an average ultimate strength of 4 293 lb. per sq. in.

Net Results.—Summarizing his conclusions from these tests, Professor Morris stated:

"1.—The action of a reinforced concrete arch rib when free from the restraining effect of the superstructure, and supported by practically immovable piers, conforms closely to the action as determined by the generally accepted theory of elastic structures, even at high unit stresses over short lengths of the rib.

"2.—The calculated compressive stress at any section of the rib checks the observed stress more closely when it is assumed that the concrete takes tension, than when it is assumed that the concrete is not active in resisting tensile stresses, even when high tensile stresses have caused cracks in the concrete.

"3.—The observed tension in the steel was, in general, less than that computed by the usual formulas assuming the concrete to take no tension.

"4.—The superstructure of an open spandrel rib arch greatly reduces the deformations of the rib; the amount of this reduction depends upon the degree of continuity of the floor system, the manner in which the floor system is attached to the tops of the spandrel columns, and the stiffness of the columns.

"5.—By the use of the Beggs deformator gauges on an elastic model of celluloid, or some other suitable material, a quantitative determination of the effect of the superstructure upon the stresses in the rib may be made, if all members are connected in a definite way. While it seems possible to get a reliable model analysis when the superstructure has frictional bearings, such as the floor bearings of the Yadkin River Bridge, there is no practical way of making a model which represents such joints with certainty.

"6.—Temperature deformations appear to be independent of the superstructure for this particular arch.

"7.—The introduction of expansion joints of the sliding type in the floor system of this bridge was not effective in eliminating the effect of the superstructure upon the stresses in the arch ribs."

Application to Arch Design.—An important point to be noted, remarked Professor Morris, was that these tests dealt only with deformations produced by large loads, and that these loads themselves were of only short duration. It would not be safe to extend the conclusions to the effect of dead loads or long continued loadings because of the possible change in the elastic properties of the concrete over a long period of continuous stress. Regarding the various conclusions reached, he went on:

"It may be inferred from the first conclusion that, even though the dead load unit stresses are kept down to a low value, the combined dead and live load stresses may be safely allowed to reach a much higher value, depending upon the quality of concrete which may be secured in the work.

"The second conclusion indicates that compressive stresses computed on the assumption that the concrete has no tensile strength, are on the side of safety.

"The third conclusion shows the same in regard to the steel on the tension side of the rib.

"Conclusions 4 and 5 indicate that, if full advantage is to be taken of the stiffening effect of the superstructure, a type of structure must be used which can be accurately represented by a model. In order to do this, sliding joints should be eliminated from the structure as far as practicable and definite connections made between all members.

"For arch bridges of unusual size or proportions, an analysis including the effect of the superstructure should be made to insure safety in the spandrel columns and in the interests of economy in the arch ribs."

Military Tests.—A forceful illustration of many of the points in the paper was given by a set of moving pictures immediately following. These showed the structure, the movable tanks, the details of measurements, the cracks, and other physical effects.

As a most spectacular conclusion of the test, the structure was turned over to the War Department and subjected to artillery fire, air bombing, and, finally, to complete destruction by mines. It is worth noting that throughout all the extreme physical tests and the military tests until the final mining, the structure withstood all efforts aimed toward its destruction.

DISCUSSION

THE YADKIN RIVER BRIDGE TESTS

At the completion of this interesting report, an unusually full discussion ensued. This was the best possible evidence of the interest of engineers in the general question and of the valuable results obtained.

VERIFYING THEORY

By G. M. Braune,* M. Am. Soc. C. E.

During his observation of the tests, G. M. Braune, M. Am. Soc. C. E., was impressed with the close agreement between the theoretical and measured vertical deflections. He emphasized that this was true in spite of a crack at the springing line that extended four-fifths of the way through the bridge. To him, the vertical measurements from the taut steel wire were most satisfactory because they were direct and offered little chance for errors. For this reason he placed much significance on the action of the arch as an elastic structure even after cracking.

Referring to history, Professor Braune compared the present test with those made about 1895 by a committee of the Society of Engineers and Architects. The early committee used arches of 75-ft. span and several different types, including brick, stone, concrete (plain and reinforced), and steel. Those older engineers were satisfied that their elastic theory—the same as that used to-day—was correct. Among other things, they made elaborate computations to determine a so-called elastic coefficient or deformation coefficient. He believed that a similar computation for comparison would be most interesting in connection with the Yadkin River Bridge tests. His one regret was that the limited time allowable for the tests of necessity restricted their scope.

* Dean of Eng. School and Prof. of Civ. Eng., Univ. of North Carolina, Chapel Hill, N. C.

ELEMENTS AFFECTING DESIGN OF ARCHES

By Hardy Cross,* M. Am. Soc. C. E.

Dependable Structure.—For several centuries, said Hardy Cross, M. Am. Soc. C. E.,

"Engineers have recognized the arch either in masonry or in concrete as probably the most dependable of all structural forms. Perhaps it was too good to be true and we are investigating it further to see if it is as good as it seems."

The arch itself, said Professor Cross, is a most dependable engineering structure because most of its stresses are produced from dead loading. Further, its theory depends on simple geometrical relations. The fact that certain deflections crop up when the arch bends is hardly an objection. Except for the very small secondary effect in rib-shortening, the arch does not bend at all. Thus, about one-half the stress is quite precisely defined. A similar dependability exists regarding the distribution of loading over a section.

Integral Deck.—Analyzing the effect of the concrete deck, Professor Cross considered this in three parts—a saddle or short column near the crown, having an appreciable effect on the live load stresses, and two long columns near the haunch and springing. The latter are flexible and do not have much effect apparently on the arch action. Thus, "most of the integral arch action can be explained as a result of the saddle which covers about one-half the length of the arch."

Still another element in the effect of the deck superstructure is its ability to carry and distribute loads by virtue of its own continuity. Thus, the effect of a load on the total stress is indeed small. The deck cannot affect dead load stress, but it almost certainly has a tendency to increase somewhat the maximum stress due to temperature.

Stresses in Rib and Deck.—Continuing, Professor Cross said:

"We find that the problem deals with about one-third the total stress in the arch, about 200 lb. perhaps in an arch which is designed for 600 or 700 lb. stress. Live load stresses are open to some debate. There has been some allusion to the economy which might result in finer design of the rib; on this count I do not think very much economy can result. If you eliminate something like half the live load stress from the rib and compute the possible economies and do some rough figuring and guessing on how much you save here in the United States in a year, you will likely find you could build three miles of concrete road with it.

"It is not a very important economic question as regards the saving which might take place in the arch rib. I think we are very much interested in the action of the rib in producing stresses in the deck, and you will need to study the design of the superstructure itself rather than the action of the superstructure on the rib.

"*Significance of Concrete Tension.*—The tests have brought out one matter which is very important. They have questioned the application of the usual methods of analysis for combined direct stress and flexure in reinforced concrete sections. They appear to indicate that the compression in the concrete in a section subject to both bending and direct stress is considerably smaller than is given in the usual analysis.

* Prof. of Structural Eng., Univ. of Illinois, Urbana, Ill.

"According to the test data a result more nearly consistent with them is given if we consider that the concrete takes tension—if we neglect the fact that it is cracked even in those cases where it is actually cracked. I suspect there is some theoretical basis for this.

"*Decks for Viaducts and for Arches Compared.*—The action of a deck in the reinforced concrete arch under change of temperature is essentially very different from that of a deck of a viaduct. In the case of a viaduct, if the temperature rises the deck expands and is subject to compression. In the reinforced concrete arch nearly all that deck will be subjected to the tendency (because of the tendency of the arch to spread its columns outward) not only to follow the deck, but to go further than the deck alone would go by its own expansion.

"However, if the deck is free to slide on the columns, the relative movement of deck and columns will be very much smaller than in the viaduct. As evidenced in the tests, it did not make very much difference whether they provided those sliding joints on their long columns beyond the saddles or not, and I believe further study will show that is quite commonly the case."

FIELD AND LABORATORY

By F. R. McMillan,* M. Am. Soc. C. E.

The excellent agreement between measurements and computations appealed especially to F. R. McMillan, M. Am. Soc. C. E., as refuting the contention that conditions in the field did not agree with the assumptions of the laboratory. He felt that the tests proved that proper assumptions of analysis would always create agreement. In other words, the theoretical and practical should always give dependable comparisons, and the failure to do so was due to improper analysis.

APPLICATION OF FINDINGS

By F. E. Schmitt,† M. Am. Soc. C. E.

Corroboration.—The valuable work performed on these tests appealed greatly to F. E. Schmitt, M. Am. Soc. C. E., as "one of the most brilliant single experiments undertaken, which had been made by public authorities in the United States." He was optimistic enough to believe that the cost would be returned in value, in some form or other, many times over.

Especially gratifying to him was the corroboration given by the actual performance of a full-sized arch with the tests on small models or laboratory specimens. He considered that the practical importance of all the various tests would be in their application to future engineering work. The most important point shown in the tests was that the various parts of the bridge, usually designed as independent elements, do actually co-operate. This is also true of other engineering structures, although engineers frequently deliberately ignore the fact to be on the side of safety.

He instanced the effect on the stresses in the chords of steel bridges due to the floor structure. Actually this is important in amount but as a practical matter is ignored.

* Director of Research, Portland Cement Assoc., Chicago, Ill.

† Editor, *Engineering News-Record,* New York, N. Y.

Spandrel-Braced Arches.—Interpreting the findings of the tests, he questioned whether concrete arch designers should not consider their structures as spandrel-braced arches. Mr. Schmitt remarked that:

"It seems inevitable that in the future we are not going to design a spandrel-braced arch bridge as we now design it, that is, as an independent arch rib merely supporting the superstructure of posts and deck, because that is a very inefficient form of structure."

Following this line of reasoning further, he thought that the spandrel-braced structure would introduce important questions of temperature stresses because of its rigidity:

"In other words, the question of substituting so highly rigid a form of arch structure as the spandrel-braced arch for the present rib arch is something that will require rather full consideration as a matter of practical design before we turn around and completely change our design practice."

REVAMPING ARCH DESIGN

By George E. Beggs,* M. Am. Soc. C. E.

In an impromptu discussion, George E. Beggs, M. Am. Soc. C. E., emphasized the implications of the fact that under unsymmetrical loading and with a continuous superstructure, the deflection is much less than that for an independent rib merely supporting the superstructure. Hence, he concluded that the external work done on the structure is correspondingly less and the equivalent internal work that must be done in supporting the loads, must also be less.

Practice Governs Dead Load.—Extending some of the previous discussion, Professor Beggs questioned why, of the total stress in the arch, two-thirds should be due to dead load and one-third to live load. He called attention to the fact that the thickness of arch rib assumed at the outset depends usually upon "rules-of-thumb" conforming to "standard practice" which differs materially in Europe and America, for example.

The dead load stress due to the arch rib alone is obviously just about the same whatever the cross-section may be. Therefore, the ratio of live-to-dead-load stress depends to some degree upon the primary assumptions. Taking the effects of temperature into account, the thick rib works against itself. The effects of temperature may be reduced by increasing the flexibility of the arch, that is by reducing the section.

"So it seems", he continued, "that in estimating what fraction the live load stress is in relation to the total load stress, it is not fair to base such fraction upon present practice."

He suggested a procedure by which the design would begin with the assumption of a thin arch rib, properly shaped, of course, to eliminate bending moment under dead load alone. In such case the addition of the superstructure would not call into play what has been termed "integral arch action" until the

* Associate Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

concrete of the superstructure had set and additional loads were imposed. Taking integral arch action into account under such procedure, in Professor Beggs' opinion, would give a much lighter structure than that resulting from the observance of current practice, and without increase in allowable working stresses.

Temperature and Expansion Joints.—As to the question of expansion joints, Professor Beggs thought that field experience would give the answer. A joint that never moves evidently is of no use. He believed that it is possible to estimate the movement of a theoretical expansion joint by models and he explained the procedure in brief. The experiments already made along this line indicate the greatest movement in the joints near the ends of the spans, particularly for bridges with low superstructure. For higher superstructures, the movement is greater, but there again a compensating influence exists in the longer columns and the greater flexibility.

The model tests showed, continued Professor Beggs, that:

"On arch ribs the temperature bending moment checks very closely with the bending moment that would be estimated by the usual theory for the bare rib. But in case we have arch ribs with continuous superstructure, the bending moment at the springing plane and also at the crown is considerably increased due to the fact that deformation tends to take place at the more elastic section of the structure. The superstructure stiffens the arch between the crown and springing with the result that the angle of deformation is compelled to take place near the crown and at the springing.

"That added temperature stress may or may not be of serious consequence. It is of serious consequence if it produces stresses that are too high—otherwise, not. So you cannot say until you investigate each particular problem as to whether the superstructure has a damaging effect on the entire structure due to this increase in temperature stresses.

"*Tests on Hinged Arches.*—In case the action of the superstructure in increasing temperature stresses should be of damaging effect, there is a possibility that design may be modified so this effect can be eliminated, possibly by using three-hinged arches stiffened by the superstructure so as to eliminate completely the bending moments due to temperature, shrinkage of the arch, and movement of the foundations.

"But, of course, such a method could not be used practically with great confidence and success until further experiments are made. We have made very few in this country and they have made a few in France, but experiments should be run, as Professor Morris indicated, on the design of hinges for reinforced concrete arches, hinges made of metal, and hinges made of reinforced concrete. Some of that type of hinge are supposed to give perfect articulation. Others are more or less of a semi-articulative character; that is, what we might call elastic hinges which develop some elastic moment but not of considerable amount."

GENERAL DISCUSSION

ARCH INVESTIGATION

Support of Arch Research.—The great accomplishment of these tests and of the work of the Society's Special Committee on Concrete and Reinforced

Concrete Arches appealed especially to A. G. Hayden, M. Am. Soc. C. E. He believed that it should be continued and extended.

For one thing he noted the investigation of spandrel-braced arches had yielded important results. A similar problem has to do with arch structures having spandrel walls, concerning which little is known. If the concrete floor system is an aid to the braced arch, presumably an integral spandrel wall would be of much assistance to the spandrel-filled arch. In the design of these, the arch rib is calculated as unaffected by the side spandrel walls, whereas the latter are usually constructed integral with the arch rib between their vertical expansion joints, if any. Nothing, thought Mr. Hayden, is known concerning this interaction of parts. The construction whereby the spandrel walls are actually separated from, and merely supported by, the arch rib is not usually followed in practice, so the problem of the spandrel-filled arch structure requires experimental investigation.

Other members in attendance seemed to agree in this matter, with the result that it was voted as a sense of the meeting "that funds be made available for the continuing of the work of the Arch Committee", which suggestion was referred to the Board of Direction.

AUTHOR'S CLOSURE

Effects on Actual Design.—Permission was given to Professor Morris to comment on the various questions raised. In connection with dead load and temperature he made the point that in actual construction the arch is usually free by the time the superstructure is built. It must, therefore, carry the dead load stress as a rib without superstructure. Similarly, the Yadkin River Bridge tests showed that the effects of the superstructure upon the deflections due to temperature were negligible. Hence, he agreed with Professor Cross that an arch should be calculated as a free rib for dead load and temperature.

In designing the reinforcement, Professor Morris thought that the concrete tension should be neglected. He pointed out that the maximum steel tension occurred at the point of cracking, and that the stress there was not measured. However, in compueting deflections or reactions the assumption of tension in concrete is proper.

Moduli and Stress Variation.—He did not believe that the use of a low value for the ratio of concrete and steel moduli was safe. When tested the Yadkin River Bridge was six years old, whereas ordinarily an arch may be loaded within two or three months, at which time the higher value is safe.

He agreed "that in unusual arch structures the bending moment should be investigated." As a final point he emphasized that the tests seemed to prove that the assumption of straight-line variation of stress is adequate.

SURVEYING AND MAPPING DIVISION

JANUARY 17, 1929, 10:20 A. M. TO 1:10 P. M.

SYMPOSIUM ON CONSTRUCTION AND LOCATION SURVEYS

At its outset the meeting of the Surveying and Mapping Division departed from the set program. Illness upset the first assigned subject completely. As a substitute, Chairman William Bowie made a most fortunate choice of extemporaneous topic—that of Construction and Location Surveys. In turn, he called on a number of members who approached the subject from diverse and interesting directions.

In opening the topic Chairman Bowie stressed the increased value of land everywhere as the most important factor in raising the standard of engineering surveys. The Division had before it the problem of relative economies in surveying, especially as applied to construction surveys.

Improvement of Instruments.—The fact that the study of surveying had been relegated to a comparatively minor position in engineering schools was deplored by W. C. Taylor, M. Am. Soc. C. E. "In an age of research", he said, "surveying has benefited but little except as applied to instrument makers in the field of manufacture." Owen B. French, M. Am. Soc. C. E., agreed with Professor Taylor that even in this respect the transit required for an accurate survey still seems too heavy and the computations necessary to close a traverse could be improved and shortened.

Airplane manufacturers are developing a radio compass. Could this instrument be adapted and perfected so that a surveyor would not need to wait for clouds to pass before orienting himself on a meridian? Are surveyors in structural and location work keeping abreast of all the possible lines of progress? These and other questions are important problems of research which Professor Taylor felt were being slighted by engineers in the general advance of other branches of engineering science.

Substantially the same sentiment was voiced by C. W. Hudson, M. Am. Soc. C. E. R. H. Randall, M. Am. Soc. C. E., also agreed with Professor Taylor on this subject of research and new developments. He expressed the belief, however, that these researches should take the form of developing and improving the ordinary every-day processes now in use.

A Manufacturer Speaks.—The reaction of a manufacturer was expressed by C. W. Keuffel, M. Am. Soc. C. E., of Keuffel and Esser Company. Manufacturers continually have difficulty in determining from surveyors what their most pressing instrument needs are. Does the engineer want a light transit or a heavy one? Does he prefer an instrument for reading angles to the nearest second or minute? In recommending the tilting dumpy level for the use of the engineer on ordinary work, Mr. Keuffel stated that,

"He [the engineer] should have a rugged instrument of the type that is simple to check and he should not have to bother about leveling up and having to adjust all four leveling screws and watch the bubble every time before he looks through the telescope."

That there are opinions extant on this subject was expressed by Chairman Bowie who replied that the trend is toward better workmanship, more accurate graduations, and better micrometers and microscopes. Professor Hudson averred that,

"When it comes to the very accurate base line work, such as we use for very long bridges, we need a special instrument. But I think you will need to have a great variety of instruments rather than a few. I do not think you can combine in any one instrument what is needed."

Will Constructors Pay for Accurate Surveys?—The question was asked whether the men responsible for financing big construction programs appreciated the necessity for accurate location surveys. Professor Hudson replied in the affirmative. For the new bridge being constructed at Providence, R. I., center-line measurements of a very marked degree of accuracy were required. In general, quoting Professor Hudson,

"Keeping track of quantities in the building of bridges, tunnels, or any public work, and in the location of the various parts of a bridge, requires a very accurate and different kind of surveying from that treated in any textbook.

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"We must get into the practice of engineering such a definite knowledge of what is required that men, when they go out to build these bridges, will have something to back them up when they make demands for platforms 50 or 60 ft. high. Surveying is a most valuable part of the whole thing, but we do not get very much of it in the schools that is really practical and the young fellow does not get an insight into what he needs."

Pros and Cons.—Very few men responsible for construction programs will pay for precise surveys, according to Mr. Randall. He believes that,

"It is not so much the fault of the construction engineer, if he does not insist on spending money for conducting surveys, as it is of the fellows who are doing their own job [the surveyors] if they do not stand on their own feet and insist upon having surveys made."

The importance of surveying to the construction engineer was acknowledged and emphasized by C. H. Birdseye, M. Am. Soc. C. E. In the construction of the James River Bridge, the contractors,

"First got the Coast Survey to assist them on the base line and then they searched for a man who was skilled in surveying practice. They got the loan of one of the best Geological Survey men available who, beside being paid an adequate salary, earned a bonus after the successful completion of the work."

Further support for the surveyor was given by remarks of Charles E. Wells, M. Am. Soc. C. E., that:

"The construction engineer has to pay a great deal of attention to the details of surveying. If he is going to make a successful record he has to work independently and lay out his base lines and do a great deal of accurate survey work. And the engineer, especially the construction engineer, who says that surveying is not an essential part of his work is 'on the wrong track'".

SURVEYS FOR BRIDGE LOCATIONS PORT OF NEW YORK AUTHORITY

By Robert Hoppen, Jr.,* M. Am. Soc. C. E., and A. H. Morrill,† Assoc. M. Am. Soc. C. E.

A paper entitled "Surveys for Bridge Locations" was read by Robert Hoppen, Jr., M. Am. Soc. C. E. The general method outlined was that used in making surveys for four bridges of the Port of New York Authority, as illustrated by the new Hudson River Bridge. The authors point out that:

"After general limits of a possible bridge site have been assumed, the problem of its most economical location involves a study of (1) the topographical features; (2) probable pier locations; (3) the most favorable highway connections; and (4) land values of property to be used as right of way. With these points in mind, data have to be secured regarding the location of streets, buildings, sewer, water, gas, electric and property lines, together with the property assessments and other incidental information.

"*Hudson River Bridge Site.*—The Hudson River at the site is about 3 700 ft. wide. On the Fort Lee [New Jersey] side there is a beach about 100 ft. wide, behind which the Palisades rise very abruptly to about 320 ft. above sea level.

"On the New York side, the ground rises abruptly to an elevation of about 100 ft., then drops into a swale and back up again to Riverside Drive, and rises then abruptly in about 300 ft. to Elevation 178.

"A fairly large area on the New Jersey side was undeveloped land; on the New York side, Fort Washington Park occupied the space from the river to Riverside Drive, beyond which the area was occupied by solid blocks of high-class apartment houses.

"*Triangulation Methods.*—A series of quadrilaterals was decided upon as offering the strongest triangulation system. Three existing Government stations were utilized. These were the U. S. Coast and Geodetic Survey Station established at the edge of the Palisades on the New Jersey side, the Highbridge Triangulation Station at Highbridge Tower, and the U. S. Engineering Department Station at Fort Washington Park on the New York side. In order to secure intervisibility, it was necessary to place many of the stations on the roofs of apartment houses. The accessibility of tying in the traverse system for the topography was also considered in the location of the stations."

As a general rule, Mr. Hoppen stated, the angles chosen were greater than 30 degrees. In reading, both exterior and interior angles were repeated ten times, five readings being normal and five inverted. Then the mean of the interior and exterior readings was taken, with a resulting accuracy of less than $1\frac{1}{2}$ sec.

Surveying Equipment.—Details of the survey were outlined by Mr. Hoppen as follows:

"Triangulation stations were permanent points marked by brass plugs or a cross-cut in the rock or stone. These were occupied by an instrument on a tripod, and sometimes by an instrument on a trivet. The trivet is more desirable because of its greater stability. When the station was not occupied,

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† Asst. Res. Engr., Hudson River Bridge, The Port of New York Authority, New York, N. Y.

a white pine pole, 2 in. square, painted alternately black and white, of suitable height, and guyed by three wires, which were adjustable, served as a means of finding the station, and to sight on. Before sighting on these rods, we always checked them for being on center and plumb. A flag of white canvas, with a black square, was tacked to the top of the rods as a quicker means of 'picking them up.'

"The instrument was always sheltered by a large umbrella during the reading of angles. This not only protected it from the sun, but also acted partly as a windshield. However, the effect of the wind was very quickly noticed, and often necessitated the postponement of the angle work."

Certain conditions were laid down, stated Mr. Hoppen, giving the basis upon which the observer could determine which angle was to be repeated. He listed these conditions as follows (see, also, the appended diagram):

"(1) First reading of interior and exterior angles to agree within 10″ of the mean of ten repetitions of the same.

"(2) Sum of the first readings of interior and exterior angles must add up to 360° plus or minus 10 seconds.

"(3) Sum of the mean of ten repetitions of interior angle, and the mean of ten repetitions of the exterior angle to add up to 360° plus or minus 2 seconds.

"(4) The sum of the field balanced mean angles about any station to add up to 360° plus or minus 5 seconds.

"(5) The sum of the interior, field balanced mean angles of any triangle to add up to 180° plus or minus 3 seconds.

"(6) The sum of the eight interior, field balanced mean angles of any quadrilateral to add up to 360° plus or minus 5 seconds.

"(7) Four corner interior angles of any quadrilateral to equal 360° plus or minus 5 seconds.

"(8) The sum of two adjacent interior, field balanced mean angles at one station in a quadrilateral to equal the corner interior, field balanced mean angle at that station plus or minus 3 seconds.

"(9) The sum of each set of opposite, balanced field mean angles of any quadrilateral to equal each other plus or minus 5 seconds."

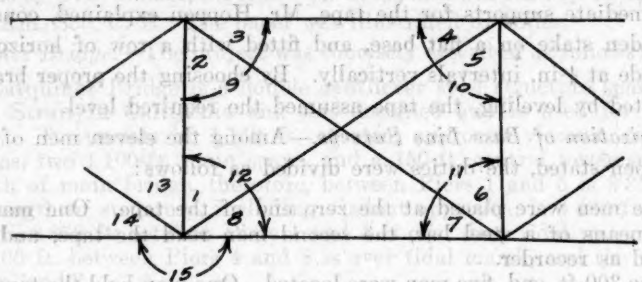
Conditions (4) to (9), inclusive, may be expressed as shown in the appended diagram and the following table:

"SCHEDULE OF REQUIRED ANGLE CHECKS.

Condition No.	Angle No., shown in diagram.	Total, in degrees.	Allowable error, in seconds.
(4)	Angles 1 + 8 + 13 + 14 + 15	360	± 5
(5)	Angles 12 + 2 + 7	180	± 3
	Angles 3 + 6 + 10	180	± 3
	Angles 1 + 4 + 9	180	± 3
	Angles 5 + 8 + 11	180	± 3
(6)	Angles 1 + 2 + 3 + 4 + 5 + 6 + 7 + 8	360	± 5
(7)	Angles 9 + 10 + 11 + 12	360	± 5
(8)	Angles 1 + 8 = 12	......	± 3
	Angles 2 + 3 = 9	......	± 3
	Angles 4 + 5 = 10	......	± 3
	Angles 6 + 7 = 11	......	± 3
(9)	Angles 1 + 2 = 5 + 6	......	± 5
	Angles 3 + 4 = 7 + 8	......	± 5

Note Keeping.—Interesting features of the field work were thus explained by Mr. Hoppen:

"In order that omissions in the field might be prevented, the observer was provided with a set of notes indicating all the angles to be turned at each particular station. A recorder makes an entry of the instrument readings as given by the observer, and when the mean of the first interior angle is figured, he gives the result to the observer that he may note Condition (1). The observer finishes the repetition of the exterior; the recorder figures its mean and gives Conditions (2) and (3) to the observer.



OUTLINE TO DETERMINE ANGLE TO BE REPEATED.

"The balanced field mean interior angle is calculated by adding or subtracting one-half the difference of the sum of the mean interior angles and the mean exterior angles from 360 degrees. Conditions (1), (2), and (3) must be applied to each angle before it is considered for acceptance. After tentative acceptance, Condition (4) is applied. It is not possible to apply Conditions (5), (6), (7), (8), and (9), until several stations have been turned. It is advisable, however, to apply these latter conditions as soon as possible as the work progresses.

"It is important before leaving a station to comply with as many conditions as the taking of the angles up to that time has permitted. This saves time and eliminates discrepancies. Any differences which develop will indicate where the trouble is, and these angles can be re-read.

"When the angle work complies with all these field conditions, the field party can then hand in their notes as a completed job. It is not then necessary to send them out to repeat certain angles and unallowable errors are eliminated before the computer starts to make final adjustments of the field balanced mean angles. Adherence to this procedure insures accurate and speedy work, and the final adjustments will then be relatively small.

"The most serious error which the field man has to eliminate is refraction, and compliance with these requirements reduces to a minimum this troublesome difference between the actual angle and that computed by instrument work."

Computations and Standards of Measuring.—To balance field-balanced mean angles:

"The sum of the angles about any point must equal 360 degrees. The difference of the sum of interior field-balanced angles from 360° divided by the number of angles involved gives the amount of adjustment. This latter balanced angle was finally balanced by the method of least squares. This gave the angle to be used in the computations.

"The base line was measured with a 300-ft. tape. The tape was standardized for 68° Fahr., 75 ft. catenary, and the tension given for these requirements by the U. S. Bureau of Standards. After the base line was measured, the tape was again checked by the U. S. Bureau of Standards.

"At the two ends of the base line being established, intermediate points were set on line at intervals of 299.5 ft. At the 599-ft. points, steel plugs were set and a fine line scratched on the head of the plug.

"This was done so that the spider would only be used at the 299-ft. point. This permitted of the moving of the spider at any time it became necessary, or in the event of its being knocked over by an automobile, the only distance lost would then be the 599-ft. measurement."

Intermediate supports for the tape, Mr. Hoppen explained, consisted each of a wooden stake on a flat base, and fitted with a row of horizontal brads on one side at ½-in. intervals vertically. By choosing the proper brad support, as indicated by leveling, the tape assumed the required level.

Organization of Base Line Surveys.—Among the eleven men of the party, Mr. Hoppen stated, the duties were divided as follows:

"Three men were placed at the zero end of the tape. One man held the tape by means of a steel bar, the second man read the tape, and the third man acted as recorder.

"At the 300-ft. end, five men were located. One man held the tape by means of a steel bar, another read the tension, a third read the tape, the fourth man recorded tape readings, and the fifth man acted as signalman.

"*Procedure.*—Two standardized thermometers, reading to 0.2°, were used, one at the 75-ft. point and another at the 225-ft. point, with a man at each to take and record the readings. There was also a man at the 150-ft. point who watched the catenary.

"When the man at the tension end had the proper tension, a signal was given to read, at which time the men at the zero and 300-ft. ends, and at each thermometer, made their readings. Two accepted sets of ten readings each were made. If the difference between the mean and any individual reading exceeded 0.001 ft., the set was rejected. Readings, of course, were made rapidly to avoid temperature changes. The notes were reduced by the usual method."

General Results.—According to Mr. Hoppen, it was found best to make the measurements at night when the temperature was more nearly constant, choosing of course a calm night. He went on:

"The New Jersey base line was tied to its corresponding quadrilateral by the use of two small triangles, one at each end. The northern triangle had for its base a part of the main base line, one point of the triangle being the end point of the main base line, and the apex the triangulation station, 'T', of the New Jersey quadrilateral. A similarly used triangle tied the south end of the base line with Triangulation Station 'H. S.'

"On each side of the river, a series of traverses was run. These covered the entire area under study. They were tied into the triangulation system at both ends. The average accuracy of these traverses was 1 in 20 000. From these traverses the physical features were obtained. All information available from city, county, State, and private maps was co-ordinated, and a final map produced.

"In all these surveys one party was placed on the triangulation system and another party simultaneously ran traverses and obtained topography. The corps consisted of a Resident Engineer, Assistant Resident Engineer, two Transitmen, one Levelman, and eight men. After the bridge sites had been finally chosen, these surveys formed the nucleus for construction surveys."

TRIANGULATION SURVEYS FOR CARQUINEZ BRIDGE, CALIFORNIA

By G. J. Calder,* M. Am. Soc. C. E.

The next paper, entitled "Triangulation Surveys for Carquinez Bridge, California", was by G. J. Calder, M. Am. Soc. C. E., and was read by A. H. Morrill, Assoc. M. Am. Soc. C. E. The paper was illustrated by slides.

Carquinez Bridge.—The project was concisely described as follows:

"The Carquinez Bridge is a double cantilever steel structure spanning the Carquinez Strait in California and was designed and is used for vehicular traffic only. It consists of 1 132 ft. of steel approach viaduct, two 500-ft. anchor-arms, two 1 100-ft. main spans, and a 150-ft. central tower span. The total length of main bridge, therefore, between Piers 1 and 5 is 3 350 ft. and the total length of structure, including viaduct approach, is 4 482 ft. Of this total length, 2 850 ft. between Piers 1 and 4 is over deep tidal water; approximately 1 100 ft. between Piers 4 and 8 is over tidal marsh, and the remainder is over dry land."

The Strait is a tidal stream with a reversing current that sometimes becomes as great as 8 ft. per sec., and the maximum vertical tidal range is about 8 ft.

"For these reasons", Mr. Morrill stated, "it was impossible to establish temporary instrument platforms between the north shore and Pier 4 which was out in the water, and the use of precise triangulation was therefore necessary."

Short Base Lines.—On the north side the natural topography was such that a measured base line of any adequate length was almost impossible to establish. Mr. Morrill explained that: "In order to obtain even a short stretch of straight base line it was necessary to establish a base line monument in water about 4 ft. deep."

On the south bank the base-line conditions were ideal and a line about 4 000 ft. long was established. Even here there was some difficulty on account of obstructions in sighting piers and measuring angles.

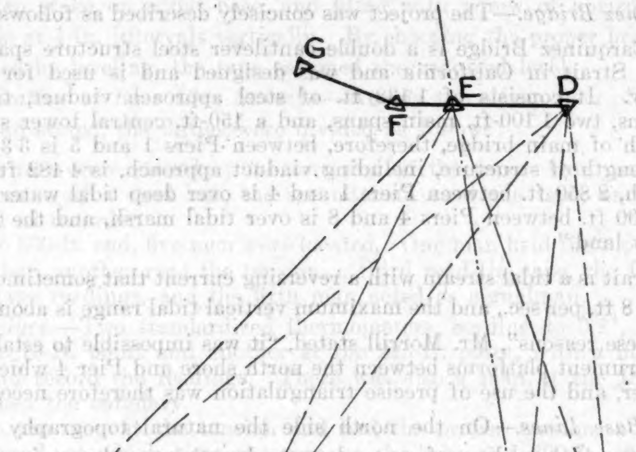
There were four surveys in all. The first was a hurried preliminary survey for the location of diamond drill borings. For this purpose comparatively short base lines were established on each shore. These were measured with ordinary surveyor's tape; angles were read with a "thirty seconds" transit; and almost no adjustments or corrections were made.

Monumenting.—For the first precise survey, permanent monuments were set on base lines along the north and south shores. These were in the shape of truncated pyramids about 3½ ft. high, with bases 4 in. and 14 in., square, respectively. This shaft was reinforced with four ⅜-in. bars, extending into the base, and wire wrapping.

The base of the standard monument was a 2½-ft. square concrete block, 1 ft. thick. The bottom of the block was 2 ft. below ground surface. In the top was embedded a 3-in. wood screw on which to mark the base-line terminals. In this way, Line AC on the south side (as shown in the diagram) was made 2 333.5 ft. In the short base line on the north side, DE was 538 ft.

* Vice-Pres. and Chf. Engr., American Toll Bridge Co., San Francisco, Calif.

Base-Line Measurements.—The second and third precise surveys were the final checks preparatory to beginning construction, Mr. Morrill explained. The methods described in the paper refer primarily to these two surveys. In the third survey, the base line, *A C H*, only was used and it was measured eleven times. For the second survey the base lines, *A C* (measured seventeen times) and *D E* (measured eight times), were used. The surveying equipment consisted of two 100-ft. and one 300-ft. steel tapes. The transit was such as to allow reading horizontal angles to the nearest 10 seconds.



ROUGH SKETCH SHOWING RELATIVE POSITION OF TRIANGULATION POINTS.

Temperatures were read at each station on a medical thermometer which was shaded during readings. Sag and temperature corrections were made on full tape lengths using the standard calibrations, and for corrections on "broken" tape lengths the Trautwine formula, $\frac{W^2\, S^2}{24\, P^2}$, was used. In the third survey the tape was supported on stakes at 25-ft. intervals and a canvas wigwam was used to shield the transit from the wind. Otherwise, the equipment and procedure were the same for both surveys.

Standards for Angle Measurements.—Quoting directly from the paper:

"In the Second Survey all individual angles, as well as the total interior angle, were measured at all stations. From one to eight sets of measurements were made for each angle and each set of measurements was repeated three times with the telescope direct and three times reversed. A 2 by 2-in. vertical stick, 10 ft. high, was set at the top of the monument as a foresight and guyed

with four wires in a vertical position. Angle measurements were taken during calm weather, preference being given to cloudy days."

A condensed statement of the results quoted by Mr. Morrill, is as follows:

"RESULTS OF BASE LINE SURVEYS

Description.	Second Survey.	Third Survey.
Mean length, in feet	Line *AC*, 2 333.503 Line *DE*, 537.999 Line *AB*, 316.60	Line *AC*, 2 333.534 Line *AH*, 3 979.296 Line *AB*, 316.594
Probable error	Line *AC*, 1 in 123 000 Line *DE*, 1 in 147 000	Line *AC*, 1 in 65 000 Line *AH*, 1 in 114 000
Discrepancies at stations, in seconds	Station *E*, 03 Station *A*, 00 Station *C*, 08 Station *D*, 02	Station *E*, 03 Station *A*, 035 Station *C*, 01
Discrepancies in triangles, in seconds	Triangle *ECD*, 08 Triangle *EDA*, 01 Triangle *AEC*, 15 Triangle *ADC*, 06	Triangle *ACC*, 04 Triangle *EHA*, 06 Triangle *ACH*, 01

Angle Adjustment.—The requirements for angle adjustment were threefold:

1.—The sum of angles at each station must equal 360 degrees.

2.—The sum of angles in each triangle must equal 180 degrees.

3.—All triangles must fit together perfectly to form a quadrilateral.

After properly weighting both surveys according to these standards the difference in the computed length of the line, *B E*, on the center line of the bridge, was 0.03 ft., or a discrepancy of less than 1 in 100 000.

Permanent Fore Sights.—What follows is described best in the author's words:

"After the triangulation system was completed and the surveys were adjusted, angles were calculated from each end of Base Line *A C* to intersect with each of the four pier centers forming the Pier 3 group. Markers consisting of 8 by 8-in. posts were set along the bluff at the north shore as fore sights. On account of the long sights and the 10 second limitation on the transit setting circle, it was necessary to take accumulated angle readings to these fore sights after setting them and then correct their positions as indicated through the increased accuracy of the angle measuring thus attained. A 3-in. vertical white strip painted on the fore-sight post could just be seen as a fringe on each side of the vertical cross-hair. The approximate length of sight was 1¼ miles. Similar sights were set on pier center lines north and south or parallel to the axis of the bridge.

"The piers were constructed by the concrete-filled timber-crib method with open dredging wells. Before the pier crib was floated to position a substantial steel and wood guide-frame with anchors was built at the approximate location of the pier. Location of this guide-frame was given by flagging into position the floating equipment, by means of simultaneous transit sights from the established points mentioned."

The matter of guiding the caissons as they settled into position was then left in the hands of instrumentmen cutting lines of intersection from the base line and inspectors who watched the tilt of the crib as the work progressed.

Final Checks.—In time, it became possible to measure the actual distance between piers with a 1 200-ft. tape, standardized with a 71-lb. pull at 68° Fahr. A comparison of measured and computed distance is as follows:

"Comparison of Measured and Computed Distances, in Feet.

	AB to Pier 5.	Pier 5 to Pier 4.	Pier 4 to Pier 3.	Pier 3 to Pier 2.	Pier 2 to *E*.	Total distance.
By triangulation........	74.00	500.00	1 100.00	1 250.00	312.83	3 236.83
By direct measurement.......	74.00	500.00	1 100.04	1 249.970	312.85	3 236.824

Merits of Triangulation in Control of Bridge Construction.—In conclusion, Mr. Morrill quoted Mr. Calder as follows:

"These figures are the result of careful field operations with legitimate corrections applied. Of course, they do not positively prove the relationship between triangulated distances and actual distances. It was observed that the long sights from base line to the pier centers sometimes varied the location of the line rod by as much as 1½ in., depending on whether the sight was taken in the morning or afternoon. It is the opinion of the writer, however, that the accuracy of direct measurement for remote piers is, in spite of all precaution, little if any greater than the accuracy obtained by a carefully prepared triangulation system, and the use of transit intersections from this system to locate the piers.

"Expediency favors, by far, the triangulation method. By this means it was possible to give the contractor his pier center at almost a moment's notice. Direct measurement required that the navigation channel be flagged by boats to prevent interference with the tape during the measurement, and the preparation for and the taking of the measurement occupied the better part of four hours. It demanded the use of six men and three towboats, while the triangulation method required three men, a line, rod, and two transits."

DISCUSSION

BRIDGE SURVEYS

By C. W. Hudson,* M. Am. Soc. C. E.

A Good Base Line.—In discussing, jointly, the papers by Messrs. Hoppen and Morrill and Calder, C. W. Hudson, M. Am. Soc. C. E., stressed five desirable characteristics of a good bridge location base line:

1—It should be at, or nearly at, right angles to the center line.
2.—Its length should be such as to give good angles.
3.—The necessary lines of sight along the base line must be open at all times.

* Cons. Engr., New York, N. Y.

4.—It should be located so as to insure that falsework or other obstructions will not interfere with its use for locating the necessary pier sights, etc.

5.—In many cases, Characteristics 1 to 4 should be recognized in drawing the contract for the bridge.

Possible Adjustments in Bridge Spans.—He expressed the belief that:

"In order to let a contract and give definite information, span lengths must be stated accurately. Where there is uncertainty as to the stated lengths adjustments in the structure itself must be provided in order to permit it to be built no matter whether the actual distance between any given pier centers is greater or less than computed, when contract plans were made. Such adjustments within limits are generally not hard to make. For certain types of bridges, however, such as arches with fixed ends any great amount of adjustment in the length of the structure would be very difficult. In a cantilever bridge considerable variation can be taken up at the ends of the suspended span. In a suspension bridge, the variation would be simple if it did not prove to be large.

"In building the Hill-to-Hill Bridge in Bethlehem, Pa., which was located in a closely built-up section of the city, in a locaton where the topography was very irregular, any actual measurement of the center line was impossible before the right of way was cleared, which is the case in many important bridges. The center line was actually established from two carefully checked and re-checked traverses. The actual length of the main stem of the bridge was 1 960 ft. This same distance later measured on the bridge proper by both the contractor's engineering party and my own showed a difference from the originally calculated length of about $\frac{7}{8}$ in. For a structure the individual units of which are quite small taking care of a probable error of such an amount is a very simple matter."

CLOSING DISCUSSION

Gratifying Results.—Mr. Hoppen agreed with Professor Hudson that in selecting a base line, the surveyor "should not worry about whether a bridge is going to be built, but rather about the contractor who is going to try to build it." He should try to establish base lines so that other base lines can be run from them. In commenting on the results of the surveys for the New York bridges, he reported that,

"We never lost a point at Perth Amboy in eighty-three piers, and on every fourth pier we set four shoes and on the others two shoes. These were located from the ground, and when the steel contractor placed those shoes at those marks, we moved but three of them."

The co-author, Mr. Morrill, discussed briefly some phases of the surveys for the Port of New York Authority. When it was desired to check the distance on the main span at the Outerbridge Crossing, at Staten Island, surveyors went through four operations, according to Mr. Morrill. Quoting further:

"We had two checked triangulation measurements to inaccessible piers; that was followed by a measurement made by the contractor, a direct measurement [catenary] after the piers were constructed, and after the steel work was up, we ran a measurement over the top of the bridge, and all four of them, involving a distance of the main cantilever span of 750 ft., agreed within less than $\frac{1}{4}$ in."

DISCUSSION

SURVEYS FOR THE HOLLAND TUNNEL

The response to the next paper, "Surveys for the Holland Tunnel", by Charles L. Crandall, Assoc. M. Am. Soc. C. E., was spontaneous. A number of members made noteworthy comments, the first of whom was Ole Singstad, M. Am. Soc. C. E., Chief Engineer of the work.

Cross-Section Control.—A number of interesting details were noted by Mr. Singstad. He emphasized the fact that the deviation of this large diameter tunnel from a true circle was no greater than on much smaller tunnels. This was due primarily to the development of a new type of high-tension steel pole. It was tightened to an initial stress of 25 000 lb. per sq. in. before the iron was let out of the shield, so that the tunnel took any external load. The result was that the diameter did not vary more than an inch, and the tunnel is unusually airtight.

He gave full credit to his subordinates for their part in perfecting this device. His words follow: "I might also say that I have been personally credited with the development of this pole", and, again, "the credit for the first suggestion", he remarked, "should be given to one of my assistants, Mr. Ralph Smillie, and the detailed development to Mr. O. L. Brodie, both members of the Society". He predicted that this pole will be part of the standard equipment in the construction of all future cast-iron tunnels.

Tunnel Distortion.—Commenting on the distortion in the tunnel, Mr. Singstad said:

"While the average deviation from true diameter is about an inch, we found that in the river section; that is, between the river shafts on the two sides, which is a stretch of about 3 400 ft., the maximum elongation of any diameter was ⅝ in., and the maximum shortening of any diameter was 1⅝ in. In the land sections, where there were more non-uniformities encountered in the ground, the maximum distortions were greater and reached as high as 3 in. in exceptional cases.

"The tunnel moved about a good deal in the river bed during the period of construction, due to the excessive pressures which were set up in the ground in driving the tunnel blind and not taking in the full quantity of muck, and we had an extreme movement of as much as 16¾ in. That was a maximum rise which occurred about 200 ft. west of the New York river bulkhead; on the New Jersey side the maximum rise was 11½ in. This rise, however, developed during the early stages of construction, and the methods of controlling that were perfected later, no difficulty having been experienced after it was thoroughly understood what caused it and how it could be controlled.

"The rise that took place in practically all of the tunnel after erection has gradually settled, so that for almost the entire length of the tunnel it did come back, prior to the opening of the tunnel, to its original position at which it was built and seems to have stopped there, with the exception of a stretch on the New York side near the bulkhead where it rose an extreme distance of 16 in., and it came back only about 25 per cent."

Precise Control Lines.—The description of the means of carrying lines and grades through the air-locks called for extensive comment by Charles S. Gleim, M. Am. Soc. C. E., who remarked that:

"Care must be taken to place the instrument or the point in that part of the lock which is encased in the concrete bulkhead. The reason for that is that you will find when you compress or decompress the air into the lock, the lock will move. I know of one case where there was an error discovered and a great deal of time and money was spent in checking up the lines to find out where the error was, and it was finally found that the point in the lock had not been set in the concrete bulkhead that moved around."

The method of carrying precise lines up into the "head", as described by Mr. Crandall, has the advantage that it can be used for construction purposes, and tunnel movements or changes in the line can be easily adjusted.

Another Control Method.—Another system which Mr. Gleim said had been used on the Hudson-Manhattan Tunnel has certain advantages. To quote:

"The precise line is a random line and the points were placed in the tunnel at convenient distances and off the center line at such locations that they could be best seen or where there was the least interference. Precise angles are turned to each point and precise distances measured, and the co-ordinates of each point computed. The co-ordinates of all the curve points and station points are computed when the alignment of the tunnel is fixed and from the leading precise points temporary construction points on the center line are placed on an offset; that is, your precise line is a random line and you place it where best you can, and you compute the co-ordinates of it, and your construction line is a temporary line with temporary points which is set off from that precise line. As the tunnel progresses forward, the precise line is re-run as often as necessary and new co-ordinates are computed and new points set ahead.

"This system has the advantage that the points by which the tunnel is guided are set from the last precise points, so that any error is not cumulative. It is of particular advantage in small tunnels or tunnels where side-drifts are used. The co-ordinate system is a distinct advantage where it is desired to have a relation between the tunnel and the surface at all places, such as occurs where tunnels pass under structures and other private rights of way. This system is also very useful when there are a number of sharp curves, as the points can be placed to the best advantage for long chords.

"As to the accuracy of the two methods, I do not think that you can definitely state. I have used both of them and both of them have been successful. I think that the accuracy depends on the workmanship and the refinements rather than the methods employed."

Practical Difficulties.—The practical difficulties encountered by the surveyors on the Holland Tunnel were further emphasized by F. A. Snyder, M. Am. Soc. C. E. Briefly stated,

"The check work was practically all done in the winter and the smoke from the Erie Yard, especially, was terrible. Night after night these men were up on the buildings trying to read angles, and very often a week would go by without any results."

Chairman Bowie expressed surprise that electric spot lights were not used in the triangulation work instead of the illuminated board. Reasons were given by Messrs. Snyder and Gleim, as follows:

1.—The illuminated board can be found most easily;
2.—It permits a greater degree of accuracy; and
3.—It cannot be confused with a multitude of similar lights.

In reply to O. B. French, M. Am. Soc. C. E., Mr. Singstad stated that the shafts through which the distance line was transferred were about 25 ft. wide.

Need of Unified Control.—To overcome some of the difficulties met, R. H. Randall, M. Am. Soc. C. E., declared his belief that:

"The preliminary studies and perhaps even the triangulation for final placement of the piers might have been to a large extent avoided if, as a preliminary, the Federal Government had had controlling triangulation positions in the city area and the city had really completed its own job of establishing triangulated points in expansion of that."

In reply, Chairman Bowie stated that:

"Eventually every city in the country will have a strong triangulation net over its whole area. It will have to do so for the various alignment surveys, for its underground and overground structures and tunnels, and so forth, and for pedestrian purposes. When that happy time arrives, I should think the work of the engineer and the surveyor on tunnel construction would be somewhat reduced."

CONSTRUCTION DIVISION

JANUARY 17, 1929, 2:30 TO 5:00 P. M.

CONSTRUCTION OF JAMES RIVER BRIDGE PROJECT

By R. C. Wilson,* Esq., and Herbert B. Pope,† Assoc. M. Am. Soc. C. E.

A most important construction project was described in a paper entitled "Construction of the James River Bridge Project" by R. C. Wilson, Esq., and Herbert B. Pope, Assoc. M. Am. Soc. C. E. This paper as read by Mr. Pope gave a short general description of the work, as follows:

"The James River Bridge project is an important line in the Atlantic Coastal Highway System and now makes it possible for a motorist to reach Norfolk from the north and continue south without long detours of troublesome ferries. The project consists of three bridges totaling 5⅝ miles in length, and about 11 miles of concrete roads built to give access to the bridges from existing highways. The largest of these bridges across the James River just beyond the city limits of Newport News, is about 4½ miles long. The other two bridges carry the highway over Chuckatuck Inlet and the Nansemond River, respectively. The traveler will now find a splendid concrete road leaving the Richmond Highway just west of Newport News and connecting with the State Highway leading to Norfolk."

Pre-Cast Concrete Piles.—Before undertaking the project, Mr. Pope explained, a very careful study was made to determine relative economies from an investment standpoint. This resulted in several changes in point of view, the most important relating to piles. The conclusion was that although many of them would have to be quite long, pre-cast concrete piles would cost less as the trestle support than any other type of bridge structure. The longest piles were 115 ft. over all and 24 in. square; they weighed 35 tons each. Many of them that were driven as much as 90 ft. in soft clay without serious resistance indicated a final carrying capacity of only 18 to 20 tons by formulas based on penetration under impact load. However, they actually supported test loads of 90 tons without settlement and are expected to carry, as the maximum future requirements, 42 tons per pile. According to Mr. Pope:

"When the estimate of making and driving these great piles was completed, it was found that the cost for carrying bridge loads averaged nearly \$9 per ton. This information led to a careful scrutiny of the design of the bridge deck so that as small a portion as possible of the carrying capacity of the piles might be consumed in supporting the dead load of the bridge. This study led to the adoption of a deck constructed of reinforced concrete slabs supported on steel I-beams in lieu of the all-concrete deck originally assumed.

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The total saving in dead load amounted to about 26 000 tons. This lightening permitted the spacing of pile bents 44 ft. on center instead of 34 ft., and saved in the cost of the piles and piers alone more than $300 000, and nearly as much more in the deck itself due to its great simplicity of construction."

Triangulation Control.—Triangulation survey work was unusually extensive for the construction project. Briefly, the various problems were solved, as follows:

"Base monuments had to be established by careful triangulation, and permanent working points had to be set so that it would be possible to start work at different locations along the line of the bridges without a chance of error of closure as each part of the work developed and approached the other. The initial triangulation work was done under the auspices of the U. S. Coast and Geodetic Survey whose men established the monuments designating the terminal points of the three bridges. No great problem was presented by the location of the roads, or even the two smaller bridges, and station points were established along the lines so that work could be started at any desired location.

"The survey work on the James River was a more serious matter as the terminal points were nearly 24 000 ft. apart with deep water intervening and poor visibility. Finally, it was decided to erect pile-supported platforms at intervals along the line of the bridge and accurately locate on these rigid platforms transit points that would be used as reference marks in starting any section of the work. The instrument point in the center was located on a platform supported on four wood piles securely wrapped and stapled together. The working platform surrounding the instrument point was independent from it, and permitted the tying up of boats without danger to it."

Operating Base.—On a job of this size the preparations necessary before actual work can begin are often problems calling for the highest type of administrative and engineering judgment. On this subject Mr. Pope stated:

"The problem of first importance in the construction of this project was the establishment of a main operating base. It was essential that this base have certain qualifications, such as:

"1.—Deep-water wharfage.

2.—Rail connection.

3.—Electric-power fresh water supply.

4.—Harbor for refuge of floating equipment.

5.—Sufficient area for storage of materials, pile casting and storage yard, carpenter shops, offices, and room for fabrication and storage of reinforcement, and many other items.

6.—Reasonable accessibility to the different parts of such a widespread piece of work.

"After a survey of available sites we were fortunate in finding one that was ideal for our requirements, and were able to lease the major part of one of the shipyards that had sprung into being during the war."

The frontage on Hampton Roads was 700 ft., with a wharfage of 1 700 ft. along a small boat harbor. A space, 1 100 by 125 ft., was set aside as a casting yard in which a total of almost 3 000 piles were made.

The Casting Yard.—In the casting yard were driven a veritable forest of 2 by 4-in. stakes which served to support platforms on which piles were cast. Side forms were braced and plumbed by means of especially constructed removable members. The piles could be made in lengths up to 115 ft. Since the

longitudinal steel came in full lengths some of the bars measured 117 ft. The mixing plant, according to Mr. Pope:

"Consisted of two 1-yd. mixers which were fed from a large charging hopper holding approximately 400 yd. This hopper was loaded by means of a stiff-leg derrick located on the bulkhead in such a position that material could be directly charged into the hopper either from the barges on which it arrived, or taken from a storage pile located under the swing of the boom. Aggregate entering the mixer was measured by batch hoppers and the water content was accurately measured by means of an approved measuring tank. Cement was brought to the mixing plant from the cement house by means of a belt conveyor.

"*Manufacture of Piles.*—After the piles had been poured for a period of approximately seven days they were lifted from the forms by means of a gantry crane which spanned the casting platform. The pulling of the piles was facilitated by heavy T-bolts with washers cast integrally. Either three or four bolts were used depending on the length of the pile. The load lifted on each T-bolt was equalized by means of blocks on the equalizing beam slung under the gantry. After pulling, the pile was carried to the curing yard which was likewise located under the crane. There they were thoroughly inspected for cracks and flaws. The T-bolts were screwed out and the holes were filled with an asphaltic preparation. For the curing yard the piles were cured for a period of 21 days or longer by means of water-jets.

"This yard had a capacity of approximately 500 piles. As piles were required they were picked up from the curing yard by the gantry crane which running out past the bulkhead, was able to place the pile directly on a barge for transportation to the drivers."

Pile-Driving Equipment.—Two pile-drivers were used, one an ordinary single-lead driver and the other a so-called "four-lead" driver. With the latter, four piles can be spotted at one time and two of them driven simultaneously with 7 500-lb. steam hammers. The driver is held in position by spuds driven into the mud.

Miscellaneous Equipment.—For concreting purposes in the field, Mr. Pope explained:

"Two barges were constructed, each 36 ft. wide by 91 ft. long. A stiff-leg derrick was installed on the rear of the barge for handling sand and gravel, from scows tied on either side of the plant, to the bin or material hopper located over the 1-yd. concrete mixer; a 65-ft. tower directly in front of the mixer hoisted the mixed concrete to a distributing hopper on the face of the tower. The cement was delivered by barges, and carried by a belt conveyor from the scow to the mixing platform.

"A water pump distributed water from tanks in the hold to various points on the barge and also supplied water to the mixer. All water used for mixing purposes was brought from shore by water barges and was obtained from tested wells or from the municipal water system of Newport News. A lighting outfit was also installed to furnish electricity for flood lights, etc. In the center of the barge a power anchor-winch connected through reduction gears to 'nigger-heads' was placed. By means of this winch the barge could work itself along the bridge as desired.

"All this equipment was gas-driven and required about 250 h.p. Wood pile dolphins were driven at intervals of perhaps 1 000 ft. along the line of the bridge. These were used for the tying up of the various equipment at night and also served as a point of attachment for the stern lines of the concrete rigs."

Cut-Offs and Field Tests.—Piles were spotted by surveyors stationed at the nearest triangulation points and the spacing was controlled by taping continuously along the line of piles previously driven. This stationing was checked from time to time by means of tested tapes. To quote further from the paper:

"All piles could not be driven to the exact elevation required, and some of them had to be cut off. This was done by means of an acetylene torch and air hammers. After the pile had been driven, certain piles showing the maximum penetration per blow were selected for testing by means of a water tank resting on top of an I-beam frame which slipped over the top of the pile. This tank was filled with water and readings were made for various loads until a maximum load of 90 tons was reached.

"Observations were taken on the settlement of the pile under each increment of load. After the maximum load of 90 tons was reached, the water was let out of the tank and readings were taken to determine how much, if any, the pile itself had been compressed. It was found that on piles which had a comparatively large penetration per blow only about ⅛-in. settlement was caused by the maximum load of 90 tons, which is about double what any pile will be required to carry.

"*Elements of the Trestle.*—The typical trestle deck consisted of pile bents supporting longitudinal I-beams upon which a 9-in. concrete slab was poured. The bent consisted of four piles, 7 ft. on centers, capped by a concrete beam bonded to the pile. The forms for these pile caps consisted of a timber yoke on each pile; on top of these yokes were laid two 6 by 10-in. timbers the full length of the bent. These timbers were likewise clamped by means of bolts to the piles and not only afforded a greater resistance to slipping under the concrete load, but also pulled the piles back into line. On top of these timbers the forms for the bottom of these caps were laid and side forms were built up.

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"The I-beams used were 30 in. by 115 lb., 43 ft. 9 in. long, resting on base-plates and anchored to the caps by heavy bolts. Provision for expansion was made by slotting the anchor-bolt holes in alternate beams."

In the deck slab expansion was provided for by means of an asphaltic felt strip cast into the concrete slab directly over each bent at the point where the I-beams came together. The deck was kept wet for 14 days after pouring and was protected from the sun by burlap.

The erection problems attending the steel truss work and its supporting piers would provide material for a much longer paper and Mr. Pope confined himself to a very brief description of this phase, which was illustrated by slides.

Summary.—In concluding his paper Mr. Pope remarked:

"The first pile was driven on the project just before the first of January, 1928, and the complete job was turned over to the owners on November 17, 1928. The equipment used in the river work consisted of more than 60 power boats, barges, dredges, and pile-drivers of various sizes. There were employed during the periods of maximum activity somewhat more than 1 000 men on the whole project.

"The fender system at all channels consisted of a fender 200 ft. long with 50-ft. flares. It was constructed of creosoted piling, spaced 8 ft. on centers, capped by a 12 by 12-in. timber; with six 6 by 12-in. whalings on the channel side. Dolphins of from 13 to 21 piles were driven at the break of the fender and at the end of the flares. The whole system was braced by a secondary row of creosoted piling.

"Standard electrical navigation lights were supplemented by emergency oil lights. An aviation beacon with direction marker was installed on the top of the north tower at the lift span to protect airplanes during night flying."

GENERAL DISCUSSION

JAMES RIVER BRIDGE

Bearing Value of Piles.—After reading the paper Mr. Pope, who was Superintendent of Construction on the work, submitted to a rapid-fire series of questions. In reply to a leading question by A. J. Hammond, M. Am. Soc. C. E., Mr. Pope stated that in some cases the load applied by the test tank caused what appeared to be a settlement of $\frac{3}{16}$ in., but after the load was removed some of that was recovered. There was some slight compression in the concrete, but the pile did not go down any appreciable amount.

Speaking of skin friction, he said that the piles tested were about 90 ft. in the mud so that there was 360 sq. ft. of pile surface exerting skin friction. It would be very difficult to determine whether this 360 sq. ft., or the 4 sq. ft. of point bearing, supported the major part of the load.

Concreting Details.—The danger of disintegration due to the penetration of water was emphasized by C. C. Williams, M. Am. Soc. C. E. The danger of this condition was anticipated, according to Mr. Pope. The piles were cast with a very carefully designed 1 : 2½ : 3 mix. The resulting concrete was especially dense and there was not an extensive amount of absorption shown in any of the tests. The reinforcing steel was placed at least 3 in. in from the surface.

Replying to an inquiry of E. S. Martin, Assoc. M. Am. Soc. C. E., as to net cost of plant for testing, handling, and maintaining piles, Mr. Pope stated that the detailed records of this cost were in the hands of sub-contractors. The total concrete for piles was estimated at about 25 000 yd. He replied to a question by L. E. Andrews, Assoc. M. Am. Soc. C. E., that of the 2 980 piles possibly 3 or 4 were broken in driving; none was broken otherwise. The piles were handled not less than 30 days after being poured.

Exploration of River Bottom.—Several methods were used in exploring the river bottom to determine the length of piles required. According to W. T. Ballard, M. Am. Soc. C. E.:

"The preliminary investigation consisted of test borings, driving of timber piles, and the driving of concrete piles identically the same as the bearing piles which remained in the job. There is no question that the latter method produced information far more valuable than that obtained from either the borings or the driving of timber piles.

"This was so much a fact that during the actual construction test piles made of concrete, of the same cross-sectional area, were driven at various points ahead of the actual work of construction, for the purpose of governing the lengths of piles delivered to the various sections along the bridge. In work of this size that fact is particularly pertinent because of the great cost of preliminary investigational work."

Control of Concrete Mixtures.—Replying to question by Chairman Barney, Mr. Pope stated that the cement engineer made his test analyses on the barge at the mixer and that the concrete foreman made his adjustments based on these analyses. An attempt was made to operate on a water-cement ratio of 0.9 which gave a mixture that was rather dry but was sufficiently plastic to be workable.

In the casting yard water quantities were controlled by pressure tanks, but because of the variable nature of the pressure available on the barges the old method of water-barrel quantities was used in every other case.

Economy of Concrete vs. I-Beams.—The question was raised by Mr. H. A. Marshall whether concrete floor-beams would not have been more economical than steel I-beams on account of maintenance. It was then stated that I-beams proved the most economical because of the reduced dead load which made possible a reduction in the number of piles. Since the cost of concrete piles was $9 per ton of dead load, the saving was at least $500 000, which would pay for the maintenance charges.

REPORTS OF COMMITTEES

Immediately following, there are published in full five valuable reports by Committees of the Construction Division. Three of these were read at the Division Meeting. The first was entitled "Hydro-Electric Plant and Equipment" and was read by Chairman W. L. Locke. The second report, read by John C. Pritchard, M. Am. Soc. C. E., is entitled "Construction Plant and Methods for Concrete Buildings". The third report, entitled "Construction Methods and Plant for Steel Buildings", was read by Chairman A. J. Hammond.

In discussion, M. S. Ketchum, M. Am. Soc. C. E., declared that these reports were extremely valuable, especially as applied to steel buildings. In Dean Ketchum's own words:

"In preparing young men for the construction industry, material of this type will be necessary before very much can be done. I think we are doing excellent work in getting all this necessary knowledge ready for them."

CONSTRUCTION PLANT AND METHODS FOR HIGHWAYS

REPORT OF COMMITTEE

The following is an outline of the activities of the Committee, which, it is hoped, will serve as a plan for future reports.

A.—Grading Operations

This is the first active operation in connection with the construction of the road, and it should be divided into the various kinds of grading as:

1.—Excavation in Earth.—This subdivision should be developed through the different methods, which should outline in detail the equipment required and the actual operation of this equipment.

Methods.—

(*a*) Scrapers.—Equipment Operation:
Develop the various types of scrapers such as wheel scrapers, slip scrapers, wagon graders, Fresnoes, etc., outlining in detail the proper operation of these units for efficient workmanship.

(*b*) Machine Outfits.—Equipment Operation:
Develop the various types of elevating graders with the necessary equipment for a well-organized outfit, which should include teams and wagons or other means of transporting the dirt from machine to grade.

(*c*) Steam and Gas Shovels.—Equipment Operation:
Develop the various types and capacities of steam and gas shovels, also showing the organization of wagons and teams, trucks, iron mules, wagons, and tractors necessary to handle the earth from the shovel to grade.

(*d*) Other methods should be developed in a similar manner so that the entire subject will be covered.

2.—Excavation in Rock.—

Methods.—

(*a*) Drilling and Blasting.—Equipment Operation:
Drills (machine and hand), explosives.

(*b*) There should be developed the proper operation of the various kinds of drills, together with complete placing operations.

(*c*) Methods of Handling.—Equipment Operation.

(*d*) There should be outlined in complete detail the methods of loading the excavated materials into the transportation units; also develop each type of transportation unit commonly used in handling rock excavation. This should be treated in a manner similar to methods of handling under "Excavation in Earth", and there need be no duplicate where the same equipment is used, but reference can be made to Item 1, "Excavation in Earth".

3.—Fills.—

Methods of handling and equipment operation of (*a*) Scrapers; (*b*) Machine Outfits; (*c*) Steam and Gas Shovels; (*d*) Drag-lines; (*e*) Industrial Track; (*f*) Other Methods. (The method of construction of fills should be developed in detail like the other grading operations, showing the equipment used and outlining the proper method of operation for handling the work economically.

B.—Plant

It is thought best to develop the plant required for each type of pavement construction, *viz.*, concrete, brick, sheet asphalt, bituminous concrete, bituminous macadam, water-bound macadam, gravel, and other types. To shorten this outline only one type will be included.

1.—Portland Cement Concrete Pavements.—

(*a*) Site.—Develop essentials of a logical plant site, such as topographic conditions, proper drainage, good switching facilities, lack of railroad grade crossings.

Discuss rental values, agreements with railroads and property owners, return of property, and such features, which have a direct bearing on the selection of a particular site.

(*b*) Layout.—This phase should essentially develop the logical layout of the several units in a plant site, such as, railroad side tracks; storage sheds; stock piles; storage bins; shops; housing facilities; field office; temporary roads.

(*c*) Equipment.—

Unloading.—Proper size and type of industrial crane, A-frame derrick, etc., including also its best location and its efficient operation.

Switching.—Proper track layout for handling the several items of work (including the method of switching cars through the yards), such as railroad facilities, dinky engines, gravity, tractors, etc.

Storage Bins.—By types, listing the requirements as to size and efficient operation of each type.

Housing.—(1) Cement sheds; (2) storage facilities for steel, paint, and other supplies; (3) shops; (4) sheds for housing trucks, tractors, and other movable equipment.

Camp Equipment.—Complete in all details, including type and style of cook shack, type and style of bunkhouses, sanitary arrangements, etc.

Field Office.—Type and necessary equipment.

Central Mixing Plants.—Details of the equipment required, including not only the mixer, but the bin arrangement, methods of handling machinery, cement, etc.; also economic arrangement for discharge to the trucks or other hauling units.

Hauling Equipment.—Types of hauling equipment, such as industrial railroad and trucks, giving complete detail as to size and weight of trucks, proper motive power of industrial railroad, type of batch-boxes, capacity. There should also be discussed the matter of layout of switching tracks in case of industrial outfit and yardage arrangement in the case of truck haul.

Temporary Roads.—From standpoint of location of plant site and public roads and necessary connections; also necessary temporary drives in and around the track.

C.—Sub-Grade

It again will be necessary to divide this main subject into various types of roads, since sub-grade construction in several types may not be the same or require the same equipment. Where there is a duplication of equipment, however, it seems unnecessary in the final analysis to detail each type, but reference can be made to the outline in the preceding type.

1.—Portland Cement Concrete Pavement.—

(*a*) Fine Grading.—Necessary equipment to get the rough grade ready to place the forms, which should properly describe such equipment as slips and wheel scrapers, which are used to move rather large quantities of earth; proper type, size, and operation methods of sub-grade machines which run on the side forms; hand-casting methods; also proper type, size of rollers, and operation of rolling sub-grade.

(*b*) Form Setting.—

1.—Form Graders.—Outline any mechanical contrivance used in connection with excavation for forms or placing of forms.

2.—Hand Methods.—Outline in complete detail method of setting forms by hand, which should show best construction practice and economy of operation.

3.—Type of Forms.—Essential requirements of a satisfactory form, and methods of handling, placing, maintaining, and removing forms, carrying them forward and preparing them to be used a second time.

4.—Testing or checking of forms and subgrade should be described, outlining the proper methods by means of templet, 10-ft. straight-edge, etc.

D.—Placing Pavement

This subject should also be developed according to the various types of pavements, treating it from the time the sub-grade is finished until the final completion of the pavement surface. Every piece of equipment used in this work should be treated in detail, as to type, size, and operation. The various construction operations should also be developed, not from the standpoint of a specification, but from that of best construction practice, including such items as equipment, organization, and sequence of operation.

1.—Portland Cement Concrete Pavement.—

(*a*) Layout.—Each paving job has some basic method of operation which has been developed by the contractor as most satisfactory from the standpoint of efficiency and economic operation. This involves mainly the sequence of work and brings into play such items as:

1.—Method of Hauling.—

(*a*) In the case of trucks, the method of handling them, that is, whether or not a turn-table is used or whether they turn round through a hole in the forms; practice of constructing a road on the shoulder for the empty trucks to use, or, in some cases, bringing in loaded trucks over one shoulder and taking empty ones out over the other.

(*b*) In the case of industrial outfits there is not so much variety of operation methods, but there should be developed the track layout from the plant to the job, discussing such matters as side tracks, turnouts, number of cranes required, booster engines, switching cars on the site of the job, etc.

2.—Handling of cement, that is, whether it is hauled to the job on top of the aggregates, or whether it is brought out in

separate trucks or cars and dumped directly into the skip or into the car and truck. This item should also include the shaking of sacks, baling, and returning to the plant.

3.—Pipe lines and water pumps, including the size of pipe; size and type of pump and its operation; facilities for using city water; type and size and operation of booster pump where required. The matters of available water supply, artificial ponds, etc., should also be treated as well as the hauling of water in tank cars and its use.

4.—Construction layout should also include the general organization on the site of the job, that is, the amount of superintendence required, the handling of several operation units, such as form setting, handling of aggregates into the mixer, mixer operations, puddling concrete, finishing concrete, curing, and pulling and re-setting forms.

(b) Construction Methods.—

Equipment.—The item of equipment should discuss and analyze the type and size of equipment required on a concrete paving job and should be complete. It should include such items as; (1) Turn-tables for trucks; (2) mixers; (3) finishing machines; (4) belting machines; (5) floats of all types; (6) small tools; and (7) supplementary mechanical devices, such as calcium chloride spreaders, form pullers, equipment for testing sub-grade, equipment for testing thickness, etc.

Construction Practice.—This subject should be developed from the beginning of a process after the sub-grade is prepared; for instance, the first thing done is the placing of the center steel, transverse bars, longitudinal rods; then the mixer is charged, the concrete mixed and discharged on the sub-grade. Before this discharge the sub-grade must be tested behind the mixer which involves the cutting down of high spots and the filling in and tamping of low spots, after which the concrete is placed, puddled, finished, and cured.

1.—Reinforcing Steel.—Painting or oiling when required; proper methods and checking for proper line and placement.

2.—Sub-Grade Test.—Proper type of templet to be used and the methods of excavating high spots and filling in low spots, together with the proper type of tools.

3.—Mixing.—Methods of charging, mixing, and discharging the concrete, stressing such details as pertain to best construction practice.

4.—Puddling.—Develop along lines of labor required and tools used.

5.—Finishing.—Necessary machinery, equipment, type and operation, longitudinal floats, long-handle floats, testing equipment, mechanical and hand belts, methods of removing laitance, final checking, 10-ft. straightedge, final belting, and other requirements for a satisfactory surface. This item should also include methods of operation and equipment used in removing high spots after the pavement is set up. These items should be treated completely, both from the standpoint of proper equipment and proper construction methods, not as specifications, but as the actual tasks of the men engaged in the work.

6.—*Curing.*—Under this item, develop the methods of curing, such as earth, straw, ponding, calcium chloride, etc., and under each method develop the equipment used and the method of operation, showing organization and most economical construction methods.

Other types of pavement construction should be developed in the same manner as Portland cement concrete, but where there is a duplication of equipment and operations, there seems no reason for a complete analysis; simply make a reference to the same equipment and operation methods in the preceding type of construction.

E.—Shoulders

Each type of pavement requires a different type of shoulder construction. Therefore, this must be handled by types.

1.—Portland Cement Concrete Pavement.—

(*a*) Equipment.—Earth-moving equipment required to rough-in the shoulders. Special machines, which may have been built to smooth off and finish the shoulders to the required cross-section, and necessarily this item must include handling of shoulders.

(*b*) Best Methods of Operation.—The complete operation of each one of the mechanical units should be outlined under "Operation", showing the organization required.

2.—Other Types of Pavement.—These types should be developed in a similar manner.

F.—Back Slopes, Ditches, Intersections

This item should be discussed either in connection with rough grading or with shoulder work and the grading equipment and organization required should be outlined in detail.

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CONSTRUCTION PLANT AND METHODS FOR HYDRO-ELECTRIC INSTALLATIONS

REPORT OF COMMITTEE

The successful handling of hydro-electric installations depends largely on the proper solution of the following problems:

1.—Progress Schedule.
2.—Water Control, Including Coffer-Dams.
3.—Construction Plant.

The purpose of this report is to discuss these problems briefly and in a general way.

More important than any of these, however, are the field men of construction who plan and "plant" the job and who are responsible for turning blue prints into realities. This is true whether they operate under the name of contractor, superintendent, resident engineer, general manager, foreman, or company executive.

Water conditions play such an important part on these operations and the necessity of taking full advantage of low-water periods may result in such large savings that a carefully worked out progress schedule is very essential. Units of completed work per day in all the major items should be carefully computed and the work laid out in such a way as to take full advantage of water conditions. There are always certain vital dates which must be met, otherwise the entire project may be delayed an entire season, adding greatly to the "interest during construction" item, and reducing revenues from operation. Great care should be used in the selection of these dates.

The progress chart must work in very closely with "water control". After the chart has once been adopted, it should be carefully followed and no item of work allowed to get behind the schedule.

Water Control and Coffer-Dams

The basis of a hydro-electric installation is proper control of water. On most rivers in the United States either the Federal or the State Governments maintain gauging stations. While the period of time over which these stream-flow records have been kept will vary greatly, they are extremely important and should be studied with great care.

A composite graph of these records will show the flood frequency for any given month and will also determine the proper heights to build the temporary diversions which generally consist of coffer-dams. While the size and height of these will depend on the quantity of water to be handled, it is very rare that it is economical to construct them to such heights as to be positively sure they will never be topped. Therefore, it is important to design and build them so that they can take a considerable amount of overtopping and not be greatly disturbed. In general, these coffer-dams are built of timber cribs filled with rock with a center clay puddle or a clay bank on the outside for water-tightness, or of cribs faced with plank only, or steel sheeting. The bottoms of the planks are sealed to bed-rock by placing a sack containing mixed sand and cement under each plank. These sacks are placed by a diver. They must

be constructed so that they will not overturn or slide. None of the members should be strained to the point of crushing. While the coffer-dam is a piece of temporary construction, it should never be considered as such, but be built with the greatest care possible.

The selection and placing of the pumping plant are of utmost importance. The plant should always be in duplicate as a break-down would be most serious both in time and money. Wherever possible, the use of electric power is recommended; but if there is any possibility of disturbance to this source of power, steam auxiliaries should be at hand.

CONSTRUCTION PLANT

Camp.—Hydro-electric developments are generally located away from centers of population, making it necessary to provide housing and food for the construction forces; and, as part of the construction job, a village is necessary to house the operating personnel after the construction period is finished. It is recommended that the permanent village be planned and built at the start so that it can be used by the construction forces and avoid the expense otherwise necessary by providing temporary houses for these men.

Most States have laws governing housing and sanitation, which must be followed. The camp should be located near the work and should include a well equipped hospital. During the operation of the camp great care should be placed on cleanliness; and good, wholesome, well-cooked food provided. No part of the work will pay better dividends than a well-planned, well-operated, and spotless camp and hospital.

Power Supply.—Whenever possible electric power should be used, and it must be free from interruption.

Transportation.—Transportation is the keynote to all hydro-electric installations. With large quantities of material to be handled in short spaces of time, it must be conceived and operated so that a break-down is impossible.

A careful estimate should be made of the tonnage to be handled; this should be divided into "external" and "internal" transportation, external being all materials which come from an outside source, such as cement, steel machinery, foodstuffs, etc., and internal being that which originates at the site, such as excavated materials, mixed concrete, and other construction materials.

The plan should be as simple and, at the same time, as flexible as possible. The tracks and equipment should be well maintained so as to avoid derailments, and if the operation is of considerable magnitude it should follow railroad practice in so far as operation is concerned. Very gratifying results are now being obtained by the use of gasoline locomotives in place of steam. On small jobs, trucking is often advisable; and in rough mountainous country inclines are often used.

Excavation.—Consideration should be given to the use of gasoline engines on construction equipment. Recent rapid strides in gas-engine practice have produced some very dependable machines which show decided economies over steam. A study should be made of each piece of equipment to determine its value on the particular operation on which it will be used.

The use of crawler shovels is strongly recommended. Consideration must also be given to crawler draglines and the various sluicing methods. Atten-

tion is also called to the size of the air-compressor plant. An almost universal error on construction jobs is deficiency of air.

Concrete Plant.—Concrete is one of the large items of expense on most hydro-electric developments. This is true even if the water for the project is controlled by earthen dams and the head is developed by canals of various forms, for there is still considerable concrete to be placed in spillways, intakes, power-house substructures, and other foundations.

The plant required to place concrete of the required density and strength in these various structures depends on many elements which require skillful evaluation of benefits and costs.

Location of Mixing Plant.—While the location of structures often renders it impracticable to use a central mixing plant, such a plant will generally show advantages in cost of handling materials and proper control of mixtures. The central mixing plant should be located on the natural route of travel of the concrete materials entering the work on their way to the structure. Topographic features should be taken advantage of to keep cost of road work to and from the plant to a minimum consistent with uninterrupted traffic on other operations.

If access to the site is up the valley of the stream being developed, the site of the mixing plant is usually at a low elevation, near the stream, and below the hydro-electric plant. If the site is reached at a high level at either end of the work, the advantage of a side-hill location for the plant, permitting the feeding of the mixers by gravity, should be considered. Where concrete is handled from the mixer by train or trucks it is usually desirable to locate the mixing plant and attendant material storage far enough from the structure to avoid the congestion usually found in connection with the foundation excavations, form work, and other operations.

Capacity of Plant.—Progress schedules should be prepared for each of the major concrete features. A combination of these schedules will give the maximum plant requirements. As the peak requirements usually occur when foundations for dam or power house have been unwatered and excavated, the plant should have a capacity sufficient to complete the structures quickly to a height above probable damage by floods.

A plant, with 1 cu. yd. of mixer capacity for each 4 000 to 5 000 cu. yd. of concrete to be placed per month of active concrete work will have a reserve capacity, under continuous operation, sufficient to place concrete in such critical periods at the required speed. In general, it is desirable in large plants to have duplicate units rather than a single mixer unit of large size. This size of unit depends on the size of scheduled pours and methods of placing. 1 and 2-cu. yd. mixers being the size customarily used.

In a few cases 4-cu. yd. mixers have been used. This is only practicable where the concrete is to be placed in large masses, but, in such instances, it may be economical since the first cost and installation of derrick or crane equipment for handling buckets containing 4 cu. yd. of concrete is very little greater than for handling 2-cu. yd. buckets. However, the disadvantages of the large units are that other mixers and equipment must be provided for placing the many smaller masses of concrete and this results in duplication of

plant. Also, concrete forms must be built stronger to withstand the shock of dumping the greater mass of concrete, and unless the concrete has great workability, additional labor is required in the forms to place it properly.

Mixing Plant Accessories.—A plant designed for proportioning materials so as to obtain uniform mixture will pay well in savings in the cement content of the concrete and in handling costs by maintaining the workability of the concrete. For this purpose the use of automatic measuring equipment is almost essential for the usual mixing-plant labor cannot be developed to the efficiency of available measuring or weighing batchers. The measurement of sand by weighing or volume measurement in water is desirable.

Where the plant can be laid out to use bulk cement there are a number of desirable features to be obtained by its use; principally lower first cost, no bag loss, and greater flexibility in changing the cement content of the concrete without changing the measuring apparatus for aggregates.

Especially on projects of considerable magnitude the saving in the cost of the expensive cement by careful study of available aggregates and design of workable mixtures may indicate the advisability of separating the aggregate into more than the customary fine and coarse sizes. The cost of providing plant facilities for handling and proportioning two or more sizes of coarse aggregate is considerable, and while often uneconomical, should receive careful consideration in designing the plant.

The use of admixtures is sometimes desirable to increase the workability of the concrete. This usually is an expensive procedure, unless the mixing plant has been laid out to permit the handling of such material, hence consideration should be given this subject before constructing the plant.

Concrete Placing Equipment.—The principal methods used for handling the mixed concrete to the forms are:

(*a*) In hopper cars, operating on tracks passing directly over the forms, and dumping direct, or through short chutes.
(*b*) In buckets to be handled by derricks or cranes.
(*c*) In buckets to be handled by cableways.
(*d*) By chuting.
(*e*) By belt conveyors.
(*f*) Various combinations of these methods.

For long dams or scattered work distribution by cars is quite general. The tracks may be placed on the ground or on a construction bridge on the down-stream side of the dam. While masonry and timber construction bridges have been used, a steel structure is usually the best.

The derrick equipment may be either stiff-leg or guy derricks. For high structures there is considerable advantage in the use of guy derricks in that they may be located either on the face of dam or between penstocks where space for a stiff-leg derrick is not available.

In a number of cases the construction bridge has also been designed to support gantry cranes or traveling stiff-leg derricks which were used to place the concrete and handle forms and other material. At the Conowingo Dam, on the Susquehanna River, concrete hoist towers were attached to the traveling derrick towers, so that the concrete could be elevated and placed through very short runs of chutes which were supported entirely by the derrick traveler.

Handling concrete by cableways is slow and advisable only for comparatively small quantities. In the placing of concrete by belt conveyor the difficulty of depositing in all sections of a form is practically identical with the difficulty in this regard experienced by the chuting method.

Chuting plants show low placing cost in high structures and on structures in congested locations where trackage is difficult to establish. The pronounced prejudice now found in some quarters against chuting concrete in hydraulic structures is due more to failure of the chuting equipment manufacturers and personnel of construction organizations properly to design and use chuting equipment than to inherent faults of the system.

Concrete can be used in massive structures with a lower water-cement ratio and less cement per cubic yard than in reinforced concrete buildings. Chuting equipment was largely designed for building work, with hopper gates, and other features of the equipment, not adapted to the less easily worked concrete of low cement content.

In laying out chuting plants construction men have failed to see that concrete could be deposited from the chutes in all parts of the forms while maintaining proper slope of chutes. With short runs of chutes arranged to discharge over all areas of a form, good concrete can be placed at low cost.

Power Station Building.—The power station buildings on hydro-electric installations are generally structural steel buildings, with walls either of concrete or brick, depending on local conditions. As time is generally the essence of a hydro-electric development a structural steel building generally lends itself better to this type of work, as it is highly desirable to have the use of the power-house crane as early as possible so that machinery erection can proceed. This can generally be accomplished more quickly with a structural steel building than one of reinforced concrete. As the erection of the building proper follows the same procedure used for other buildings of this character, no further discussion of it will be made in this report.

Machinery Installation.—The machinery erection program is closely allied with the construction of the power-house structure itself. Consequently, in planning the program, it is necessary to work out a detailed schedule that will minimize the interference between the various job activities and permit rapid progress on the construction of the whole power station. On this account the erection of each unit is broadly divided into stages as follows:

(1) Draft-tube and discharge ring.
(2) Speed ring, scroll case, and pit liner.
(3) Runner and wheel-case.
(4) Generator, stator, and rotor.
(5) Governor, generator bearings and auxiliaries.

These schedules are carefully dovetailed into the construction schedule of the power-station structure, and particular care is taken to avoid interference in the erection of adjacent units.

W. L. Locke, *Chairman*,
L. V. Branch,
E. C. Macy,
Committee.

CONSTRUCTION PLANT AND METHODS FOR CONCRETE BUILDINGS

PRELIMINARY REPORT OF COMMITTEE

The Committee on Construction Plant and Methods for Concrete Buildings has held a meeting once a month since its organization and has canvassed a number of problems connected with its assignment. It is endeavoring not to duplicate the excellent work of similar Committees of the American Concrete Institute, reports of which were issued in 1921 and 1924, nor the work of the Committee of Methods on Plant Rental and Cost Keeping of the Associated General Contractors of America.

It has, therefore, frankly taken the position of interrogation, feeling that concrete plant and the placement of concrete is a part of a rapidly changing art. It feels that questioning practically everything done to-day, is more apt to awaken a progressive spirit in Society membership than to take the position that the problem is static, and then merely detail its position.

Mixing Plants.—The following discussion of the items related to plant is a condensed record of the opinions expressed at the several meetings of this Committee.

One of the first considerations is that of the concrete mixer, the heart of the plant. Will the concrete be mixed on the job or delivered to the job in trucks from a central mixing plant? A decision on this question involved at first glance the cost per cubic yard of job-mixed concrete *versus* the price per cubic yard of ready-mixed concrete delivered. The arguments for ready-mixed concrete, other than cost, are: Improvement of quality over ordinary job-mixed concrete due to refinements in controlling water and proportioning aggregates; reduction of ground space required at the job due to elimination of stock piles and mixer; and simplification of contractors' organization through the elimination of a trained mixer crew.

The contractor should be assured that the delivery of concrete to the job will be such as to insure a steady supply so that his placement gang can operate at maximum efficiency. The delivery equipment should provide concrete without segregation of aggregates and in such a condition that it can be placed without any increase of labor over that required for job-mixed concrete. Improvements in delivery equipment have reached a point where concrete of almost any consistency can be brought to the job without segregation, even with mixtures of high water content.

The contractor's decision for or against central-mixed concrete requires a thorough knowledge of his costs of producing job-mixed concrete, and the ability of the producer to deliver satisfactorily the proper kind of ready-mixed concrete.

Conveyance.—Whether the concrete should be conveyed vertically by metal or wood towers or masts, vertically by bucket conveyors, or at an angle by belt conveyors, are points for consideration.

The engineer's specifications for the concrete will no doubt affect the choice of equipment for lifting and distributing it to the forms. When

concrete is specified and designed on a strength basis, a low water-cement ratio or, in other words, a stiffer consistency will be used for the sake of economy; and the contractor will consider the advisability of using buggies or belt conveyors for transporting the concrete horizontally.

Contractors know the cost of erecting both timber and steel towers and the cost of placing concrete by the chute method. They also know the limitation of this method when dryer mixes are used. It would be well to investigate the cost and convenience of using horizontal or inclined belt conveyors. There should be a small saving at least on the height required for towers. This proportionate saving would be greater for low buildings than for high ones. It may be found that conveyors can be moved with less time and expense than chutes.

Concrete Mixers.—The size and number of mixers for the job are often determined by the kind of mixers available at the time of starting the job. It is sometimes false economy to let this be the ruling influence. The particular job should be studied from the standpoint of the number of cubic yards of concrete per floor, the kind and capacity of the hoisting and handling equipment and maximum and minimum (and, also, average) amount of concrete the contractor expects to handle each day.

Concrete Forms.—In spite of the refinements of mechanical details the item of forms is still the most expensive factor of concrete placement. Thus, if forms cost from 20 to 25 cents per sq. ft. for material plus the labor of their fabrication, erection, removal, and cleaning, and the concrete which they support costs 30 cents per cu. ft. in place, or 15 cents per sq. ft. 6 in. thick, it would seem that a thorough study of form work is in order, to reduce the cost of finished concrete work. Forms of steel, sheet metal, wood, pre-cast concrete, and other materials, should have further consideration.

Metal forms are justified only where they can be used a sufficient number of times to reduce the cost below that of wood forms. For this reason some adjustable device is almost necessary, as every job requires different dimensions whether it be columns, beams, or floors. Wood forms are usually a total loss after use on any one structure while steel forms may retain their value much longer. On the other hand the metal forms must be cleaned, transported to the home yard, and stored. This is a considerable expense which must be weighed in the balance as between wood and metal forms.

The ideal form would be a part of the finished structure, so that the material which makes up, and the labor which erects, the forms would remain a permanent asset. Several systems have been tried out with more or less success, but so far none of them has been economical to a degree that would warrant their general use.

At present, there are several methods of constructing concrete floors without forms. One of these, a combination of light structural steel and concrete which is self-supporting, seems to be meritorious. In this system the steel I-beams are designed to carry only the dead load while under construction. The added concrete with reinforcing steel carries the live load with a factor of safety.

The use of early strength cement permitting the quicker removal of forms is a European practice rapidly becoming prevalent in America. Increased use may be found for power saws, both portable and hand, also, for electric and air-boring machines and wrenches. Clay tile forms, metal pans, wood ribs, corrugated iron or asbestos, celotex, wall board, precast cement forms, hollow-spun precast columns, reinforced paper, and wood dipped in paraffin oil should be considered. The use of light weight materials with either clay aggregate or aerated concrete may reduce the cost of form supports.

Concrete Finish.—In order to insure dense concrete and smooth concrete surfaces continued study is necessary to secure good results.

The use of pneumatic hammers, admixtures, more cement, and more finely ground cement needs consideration. Frequently the cost of patching equals the cost of placing the concrete.

The more general use of properly proportioned concrete will reduce patching to a marked degree. The lack of water control on large floor areas has been responsible for much over-time paid to cement finishers. An excessively wet concrete cannot be finished as soon after placing as a concrete containing just enough water to make it workable; also the dryer mixture gives a much smoother finished surface, which requires less patching and rubbing or grinding.

Grinding instead of hand troweling on floor surfaces is being tried. Using vertical surfacing machines and washing surfaces with acid and other materials are also frequently tried.

Color in Concrete.—The use of color in concrete work is being considered more each year. There seems to be an enormous possibility in this practice. If architects desire color and decoration in their design, it would seem worth while to study the use of color pigments directly in concrete rather than facing it later with brick or terra cotta.

Concrete Reinforcement.—In order to reduce the cost of concrete reinforcement in place, mill bending, standardized panel lengths, and the rolling of bulb, or other special, shapes should be considered. In order to insure against oxidation of the steel, dipping in cement paint before shipment or placement might be attempted.

Combination reinforcing with hangers, chairs, and spacers needs real investigation and offers opportunities for invention. Comparison should also be made in concrete materials between structural steel fireproofed, reinforcing steel construction, or a combination of the two.

Construction Joints.—Greater consideration should be given to detailed field work of construction joints and the use of cement grout, metal strips, keys, etc. Beveled strips in forms prevent sharp edges.

Conclusion.—In view of the fact that American cities are now being rebuilt nearly every twenty-five years, consideration should be given to just how much "permanency" concrete structures should have. Should the exterior walls and floors be "permanent" and the partitions removable, so that the plumbing, heating, lighting, and partitions could be remodeled at minimum expense to suit the requirements of each generation?

It seems to be a sad but true commentary that the development of concrete plant and equipment has largely been made by sales organizations pushing their respective products, rather than by engineering research as a whole. It would seem timely, therefore, that the members of the Construction Division should give this matter their best continuous analytical thought, in order to obtain the maximum quality at minimum cost and within the least time.

Appended to this general report is a special report on "Central Mixed Concrete for Buildings", by Harry F. Thomson, M. Am. Soc. C. E., a member of the Committee. The Committee urges members to criticize these outlines. It welcomes any suggestions, which may be sent to the Secretary of the Society.

A. P. Greensfelder, *Chairman*,

J. G. Ahlers,	H. E. Frech,
Louis N. Beals,	R. L. Rinehart,
W. C. E. Becker,	E. O. Sweetser,
G. G. Black,	Harry F. Thomson,
H. M. Cryder,	L. R. Viterbo,
M. C. Emanuel,	F. C. Woermann,

Committee.

APPENDIX

REPORT OF CONCRETE BUILDING COMMITTEE ON CENTRAL-MIXED CONCRETE FOR BUILDINGS

By Harry F. Thomson, M. Am. Soc. C. E.

One of the important recent developments in methods of concrete construction is the introduction of central-mixing plants in cities of 50 000 population and more. The construction work served by such plants has varied in different localities, but, in general, it has ranged from simple structures, such as pavements and small foundations, to large foundations and concrete buildings of all types, and to municipal work, such as paving and sewer structures. Some of the factors contributing to the growth of this practice, as well as some of its limitations in building, are noted herein.

Improvement in Quality.—Probably the feature of central-mixing which is most fundamental in justifying the operation is the opportunity for improving the uniformity and quality of the concrete. The general recognition of such improvement will ultimately result in changing present bases of design, either by using a mix that is leaner, but has more uniform strength, or by using a smaller quantity of the mix now specified, because an assured higher unit strength permits reduction in area to obtain an equal strength. Either development means a decrease in the total cost, with the attendant opportunity of extending the use of concrete.

Other Features.—During the development stage, however, or until the more general recognition of the improvement in quality, several other features will justify a growing use of this practice:

(1) Convenience to the contractor, both through ordering as needed instead of planning stocks in advance, and through permitting concentration of attention on the placing and other details instead of on the mixing.

(2) Simplifying the contractors' organization by eliminating the mixing crew, and also by ordering a single item which is used as fast as received, instead of storing three or four separate items.

(3) Dispatching: The modern large central-mixing plant has a truck "dispatcher" and route supervisor, thus permitting proper dispatching and desired deliveries at the construction site.

(4) Speeding up of work (except when the construction work is adjacent to a railroad siding), because concrete can be supplied faster than the contractor can mix it, owing to the wider range and increased flexibility of delivery, taking into consideration the temperature, distance, and time.

(5) Minimizing interference from weather; for instance, the central plant usually can furnish heated concrete in winter.

(6) Economy in ground space by eliminating the storage piles; this is especially important in congested districts where interference with traffic, through utilizing street areas, is serious.

(7) Saving in total cost, in many instances, dependent on individual circumstances.

Refinements in Control

Among the possible refinements in control which contribute toward securing stronger and more uniform concrete, may be mentioned:

Water Control.—A permanently installed central plant can install equipment to insure positive, accurate, and automatic control of the water content of the concrete. Such control of course must allow for moistures of the aggregates, which is entirely feasible in a central plant. The influence of the human element can be all but removed; the use of a pressure tank with automatic measurement resulting from a dial, or other mechanical setting, is much more accurate than an open tank with visual gauge which must be watched by the operator.

Grading of Aggregates.—A central-mix operation is in position to obtain the co-operation of aggregate producers toward furnishing well-graded aggregates instead of material preponderant in some sizes and deficient in others, which are sometimes furnished when the inspection on the job is not very rigid, or the requirements are not definitely specified. The importance of the grading, as affecting strength, density, and workability of the concrete, is frequently overlooked.

The aggregates in many districts do not make the best concrete when used in the old, accepted proportion of 2 parts coarse to 1 part fine. Local regulations, which recognize the old 2 to 1 idea only, should be changed.

Measurement of Aggregates.—A central plant is also able to adopt refinements in the measuring of aggregates which would be possible on only the largest of jobs. Almost without exception they weigh sand, and many are also weighing the coarse aggregate in lieu of volume batching. The best practice apparently provides a separate scale for each aggregate. In providing weighing equipment, each scale should have a dial or other direct-reading attach-

ment for indicating the net weight in the hopper, instead of depending on the balancing of the beam; faster and more accurate weighing will be obtained by such means.

Mixing.—Mixing with large batches is more accurate than with small. Also, the central plant can install meters to insure a certain minimum mixing time, commonly as high as 2 min., instead of the haphazard ½ min. to 1 min. which is usual with portable job-mixing. Such timing limits the output from a given mixer, but contributes to a stronger, more uniform concrete.

Specialization of the Personnel.—In a permanent central-mix operation skilled workmen contribute in many small ways toward improvement. This specialization is reflected in the refinements in the mixing of concrete, which has now developed to the status of a manufactured product. Formerly it was regarded as a more or less irregular mixture of cement, aggregates, and water; this was owing to the irregular inspection at the mixer and to the uneducated and irregular personnel engaged in the mixing.

Central-Mixing for Buildings

The two questions which are most frequently raised in discussing the practicability of applying in building construction the improvement in quality offered by central mixing, are: (1) The influence of the time of haul on the strength of the concrete; and (2) the method of transporting the concrete from mixer to job.

Strength vs. Time of Trucking.—A widespread misconception is held by many experienced contractors, that the strength of concrete is impaired if not placed in the forms within a comparatively short time, not exceeding 30 min., after being mixed. This idea is reflected in the building codes of many cities and specifications of many architects and engineers. The fact is frequently overlooked that the initial set of modern Portland cement does not occur under normal temperature for from 2 to 3 hours after mixing. It has also been shown by repeated observations that the re-handling of the concrete during this interval does not reduce, but may decidedly increase, the strength. The one factor which is sacrificed during the interval is some part of the original workability—the degree depending on the conditions.

Tests of concrete after trucking, by the United States Bureau of Public Roads,* indicate increasing strength of concrete after 2 hours and 10 min. elapsed time. For the additional hour, or up to 3 hours and 10 min., the concrete showed no great reduction in strength, but might not have satisfactory workability. Similar tests by Columbia University Testing Laboratories† indicate practically the same results.

Thus, the conclusion of those who have investigated this feature has been that the strength is not reduced, but under most circumstances is increased due to better hydration, by a considerable interval, certainly up to 1 hour or more, between mixing and placing. It is also recognized that there is a decrease in workability during such interval, and that the decrease is less if the concrete is agitated than if it remains quiet in the body during the haul.

* *Public Roads,* December, 1921.

† Rept. No. 1778, December 16, 1927.

With pneumatic-tired equipment, as now used by most commercial central-mixing plants, it is possible to deliver over an area of roughly 10 or 12-miles radius during a 1-hour truck haul. Usually, the length of haul will be limited by conditions of relative costs, rather than by the physical condition of the concrete.

Method of Delivering Concrete.—The second query, that of the method of transporting the concrete from mixer to job, is probably receiving more attention from those interested in central-mixing operations than any other single feature. The ultimate and ideal type of body has not yet been produced. The truck body adopted by several prominent commercial plants is the round-bottom or "bath-tub" type of rear dump, with high angle of elevation. The experience in St. Louis, Mo., has been that a 1 : 2 : 4 mix with 6 to 6½ gal. of water per sack can be hauled 3 miles in this body with only minor delay due to a part of the load packing and not discharging itself.

At the present state of development, mixes with higher water contents appear to require some method of agitation during haul; and in several cities construction requiring fairly wet concrete is being served by "mixing trucks" which receive dry batches at the proportioning plant and do the mixing either just before or just after arrival at the job. If central-mixed concrete is agitated during haul by blades or a special rib, there is left next to the shell of the body a layer of untouched material which must be scraped out by hand, so that little gain is accomplished over the open, round-bottom body. A body which is substantially a revolving cylinder in trunnions has also been used, but this involves mechanical complications and delays in both loading and dumping. Other efforts have turned to bottom-dump hoppers, and to bodies in which the sides move either vertically or horizontally with reference to the flat bottom of the body. The application of such types has usually been in special work rather than in general commercial service.

Apparently, the best results have been obtained by dumping the truck load into a hopper, from which the concrete can be drawn as desired into buckets, buggies, hoist skip, elevator, or other container for rehandling. By use of a hopper there is retained any small quantity of water which may have come to the surface during trucking, and there is a natural remixing as the load passes into and out of the hopper. Concrete containing 6 to 6½ gal. of water per sack will not flow in long chutes, but is suitable for distributing by buggies and will place readily around reinforcing. Cylinders from a recent delivery of concrete of this consistency to a building job in St. Louis showed an average strength of 2 000 lb. in 7 days.

Precautions in Using.—The successful use of central-mixed concrete by a building contractor requires good co-operation between the contractor and the concrete vendor. In this case "concrete is not concrete" as usually known by the contractor; it is mixed on call at a distance from the job, is placed by a different organization from the one that mixes it, and, above all, it is a perishable commodity. If, however, both factors will discuss in advance the requirements of the job, ideas can be exchanged, and the mix and handling arrangements can be set up to serve the job with mutual satisfaction.

Two "don'ts" should be observed: First, the placing crew should not be permitted to add water indiscriminately, as this destroys the control through which the central mix is endeavoring to furnish a quality concrete, and also the added water frequently washes the mortar from the gravel and thus induces honeycombing and weak spots; and, second, the crew should make a point of permitting trucks to dump promptly on arrival, as delays are costly to the vendor through loss of working time of the equipment, and are also costly to the contractor because of the loss of workability when the concrete is held quiet in the truck for a considerable time.

This is another way in which close co-operation between contractor and vendor will be mutually helpful. When a contractor first uses central-mixed material, he should not condemn the operation if the first load or two does not work just as smoothly as he had anticipated, because the delivery of concrete should be recognized as a new development in the construction field, and there are numerous instances of the rate of placing by a given crew being almost doubled as the men became more experienced with the characteristics of the service.

Viewed in a broad way, the central mixing of concrete offers an opportunity of obtaining in building construction the manifest improvements in the quality of the concrete which result from more accurate control of the mixing operation than is usually practiced on the job.

CONSTRUCTION PLANT AND METHODS FOR STEEL BUILDINGS

REPORT OF COMMITTEE

Stiff-Leg versus Guy Derricks.—The preference for guy derricks over stiff-leg derricks is well warranted largely because of numerous disadvantages of the latter. Some of these disadvantages may be summarized as follows:

(1) The initial setting in the basement requires a great amount of counterweight for each leg and correspondingly large foundations.

(2) The space occupied is about 25% of the area needed for sorting and storing steel.

(3) It can erect over only 75% of the area unless the boom is "dipped" under the legs which besides being a difficult operation requires that all steel to be used be placed in the small angle between the legs.

(4) Raising it from one tier to another is a tedious and costly undertaking, being perhaps less tedious but more costly when two derricks are used, one to raise the other.

(5) To permit raising it, a portion of the steel must be left out so that the boom can be landed above.

(6) The average time required to raise a stiff-leg derrick with one gang is 8 hours.

(7) The initial setting of a stiff-leg derrick requires a week or more, depending on the method of counterweighting.

On the other hand, stiff-leg derricks can be used to advantage on high buildings surmounted by towers. Here, they can be placed on the main roof to hoist steel from the street so that it may be sorted and placed near a guy derrick which erects the tower. Using the guy derrick to hoist the steel from the street would be a slow method and would require an unusually large drum on the hoist to take care of the long wire. If two guy derricks are available, the method, of course, would be speeded up, but the disadvantage of the large drum would still be present.

Step-by-Step Method of Erecting a Guy Derrick.—Locating and placing the anchor slings of plow-steel wire constitutes the first step in erecting a guy derrick. The slings should be buried 6 to 8 ft. in the concrete of the foundations and should project about 3 ft. above the finished top. Six to eight slings are required for the actual guying and two others for use in erecting and raising the derrick. One of the latter is used as an anchorage when lifting the boom during erection; the other is placed close to the mast and used for a lashing that will prevent the boom from sliding ahead when it is raised for the first time.

The hoist, usually a 100-h.p. double-drum electric machine with a line pull of 9 000 lb., at a drum speed of 350 rev. per min., may be placed either in the basement or at ground level outside the building.

The boom sections should be bolted together on skid timbers running from the street to the basement, the bottom end of the boom being placed on a timber which rests on a mat of heavy planks at the proposed mast location.

The hoisting wire and sheave blocks that eventually will be used as the boom lift should next be stretched from the top of the boom. Then with the bottom of the boom lashed to the auxiliary anchor sling and with temporary guys attached to the boom, the hoisting wire is made fast to the drum of the hoist and the boom raised to a vertical position. When the inclination of the boom at the start is not very great, it will be found advantageous to place a vertical pole at the base of the boom over which the hoisting wire can be run; this makes the line pull more effective.

With the boom in a vertical position the falls can be used to assemble the mast and place the foot-block; and with the guys fastened to the mast it can then be raised to a vertical position and the guys attached to the anchor slings and equalized by turnbuckles.

As final operations, the boom is connected to the mast by a pin and the temporary guys used on the boom are tucked inside it for use when the derrick is raised. On an average, the erection of a guy derrick requires ten men three days.

Step-by-Step Method of Raising a Guy Derrick.—The most difficult procedure in raising a guy derrick occurs during the first move upward from the basement. The following steps are necessary:

(1) Place two 20 by 20-in. timbers on the beams of the floor to which the derrick is to be raised, one timber on either side of the mast.

(2) By means of a foot-block raise the boom enough to permit removal of the pin.

(3) Turn the boom 180° thus placing the hoisting block next to the mast, and steady the boom with the temporary guys which have been stored inside the boom.

(4) Attach the hook of the hoisting block to a lashing placed around the mast at a predetermined elevation, at the same time fastening the foot-block to the mast with turnbuckles.

(5) Release the mast guys, hoist the mast and foot-block to the 20 by 20-in. timbers, and transfer the guys to anchor slings that have been attached to column tops spaced about equally and with due regard for the area which will be required to land the steel hoisted from below.

(6) Release the temporary boom guys, raise the boom, and insert the foot pin.

The time required to raise a guy derrick is 2 hours, although the first move up from the basement usually takes longer. On all successive raises the guys are placed on the same columns unless the size or shape of the building changes.

Step-by-Step Method of Lowering a Guy Derrick.—Briefly, the lowering of a guy derrick is accomplished by using a boom as a pole to lower the mast two stories and then to dis-assemble the mast with the boom. The boom is then taken apart and the sections of both the mast and boom are lowered by means of a "Chicago" boom—a heavy timber fastened to a foot-block and using one of the building columns as a mast. An alternative method is to place a heavy timber extending over the edge of the building on the floor above the one where the derrick was taken apart. By means of block and tackle attached to this timber the derrick and other materials may then be lowered.

Guy Wires: Their Sizes, Location, and Importance.—The minimum size guys on derricks up to 10-ton capacity should be 1-in. cables of plow steel, with 6 strands of 19 wires each. On 25-ton derricks wire guys of 1⅛-in. plow steel should be used, while on derricks of 25 to 50-ton capacity the minimum size should be 1¼ in. Eight guys may be attached to the steel spider on top of the mast, although, unless the full capacity of the derrick is to be used, six guys usually will prove sufficient.

The spacing of the guy anchors is affected by the point chosen for unloading the steel from trucks. If heavy pieces are to be erected, two or three guys should be placed directly opposite the point at which the heavy load is to be picked up.

It is desirable to place all the anchors equally distant from the mast and all at the same elevation, although such a layout is sometimes impossible.

Adjustment of the guys should be approached with unusual care, especially when they are unequal in length or when some of the anchorages are below the level of the foot-blocks. The only practicable method of adjustment involves shaking or surging the guys, one after another, and adjusting the turnbuckles as seems to be necessary. When booming out extra heavy loads, the boom should be lowered only a short distance at a time and the guys tested and adjusted after each boom movement. Finally, it should be remembered that the greatest possibility of failure of equipment occurs not when hoisting, but during lowering when the impact of a suddenly stopped load may strain the cable beyond the breaking point.

Handling the Steel.—In hoisting steel from the street to the upper floors, it is desirable to take it directly from the trucks. It should arrive in the trucks in piles of about 10 tons, separated by means of 6 by 6-in. timber loading blocks, thus permitting the hoisting slings to be attached easily. The sling used in hoisting work is termed a bridle and consists of two slings of 1 or 1¼-in. steel wire joined by an eye which engages the hook of the hoisting block. On the ends of each wire are eyes and loose pin-screw shackles, the pins being removed and replaced each time a load is picked up. The use of two bridles is advantageous inasmuch as the ground men may be attaching one while the load is being hoisted.

It is common practice to hoist all columns first, followed by the spandrel beams and wind-bracing, and, finally, by the interior beams. An entire tier consisting of the steel for all floors between column splices is hoisted before any erection is begun.

On the working floor, the steel should be sorted and placed in the panel over which it will be erected. The outside columns and wind-bracing are erected first. A corner panel is then erected and plumbed and interior erection continues from this point. Erection always progresses from the outside toward the derrick. As soon as one panel across the entire building is erected, wires are stretched to bring the structure into plumb. Transits are not necessary for this work, the results with a plumb-line being sufficiently accurate. Elevator constructors consider that shafts are satisfactory if they are not more than ⅜ in. out of plumb.

Great importance attaches to the immediate riveting of the top floor of a tier, because (1) riveting eliminates complete and rigid bolting of the working floor which supports the next tier of steel; (2) the temporary flooring, therefore, will not have to be disturbed by the riveters, who follow several floors below the erection assembly; and (3) a riveted floor gives the derrick additional stability.

Under ideal conditions the tile or concrete floor construction should follow steel erection closely, thus affording complete lateral stability of the structure at the earliest possible moment. In case this is not advisable the building should be braced temporarily by adjustable diagonal guy wires, extending from the top floor down two or more stories. In spite of this latter precaution it will often be noticed that the building weaves considerably when the boom falls away from the mast in order to hoist a load of steel from the street. This is an added reason why unusual care should be exercised in operating derricks on tall steel building frames.

Temporary Planking.—Primarily a safety measure, the temporary planking of floors has developed into an economical necessity largely by raising the efficiency of the workmen. Three-inch plank may be used over the entire floor or 2-in. plank in combination with 6 by 8-in. timber joints. The cost of either type is about the same.

Temporary flooring should not be called upon to carry loads heavier than full rivet kegs. The major loads of steel should be carried by four planks, one on top of another, placed directly over the floor-beams. In up-ending columns from a pile on the working floor care should be exercised to place the end over a beam.

Planking is handled from one floor to the next by the hoisting line outside the building. Care should be exercised in piling the planks so that a single sling will hold them securely. As an aid to safety in handling, undressed planks of a single size should be used. On the top of the tier of steel the plank should be landed so as to reduce rehandling to a minimum when laying the new working platform.

Care should be exercised to protect the plank from high winds. The new steel, of course, will hold the planking in place as soon as it is landed, but it is sometimes advisable to wire the plank in position as an added precaution.

Signaling Systems for Hoist Operators.—In general, there are two systems for signaling the hoist operator. One uses an electrical push-button and the other, a bell cord. Signals are as follows: One bell to raise, one to stop; two bells to lower, one to stop; one bell to turn left; and two bells to turn right. The bell-cord system has an advantage over the electric signal in that the hoist operator can time the signal man's next move by the observed tension on the cord.

Protection should be afforded bell cords by placing them in a wooden enclosure; otherwise, it is possible for them to be accidently pulled. Electric wires should be encased in rubber up to the working floors, when they should be further protected by encasing in a rubber hose.

Personnel.—The personnel on a typical job includes (1) a superintendent; (2) a timekeeper who is also material checker; (3) a foreman for each der-

rick, with a gang consisting of seven men—hoist operator, swing-line man, signalman, two connectors, and two floormen who hook on material; (4) a rivet foreman; (5) a foreman in charge of such detail as placing small pieces and correcting shop and drawing-room errors; and (6) a foreman in charge of planking. The rivet gang usually includes four men—a heater, bucker-up, passer, and riveter. The riveter and passer alternate, each working 4 hours.

Wood vs. Steel Hoisting Towers.—The steel tower has many advantages over the wood tower for tall steel building erection. The total cost of erection, maintenance, and dismantling is much less for a steel than for a wood tower. The joints of a steel tower are more rigid and safe. Damaged members in a steel tower may be quickly removed and replaced. Finally, the total weight of a steel tower may be concentrated at its base, while with wood towers it is often necessary to place bearing supports at various floor levels, which interfere with the construction of the building. Steel towers are usually furnished on a rental basis and erected and dismantled by an organization thoroughly insured.

Hoisting Equipment.—Electric power for hoists, with few exceptions, has supplanted steam and gasoline in tall steel building construction work. This largely has been brought about by the rapidity of improvement in electrical equipment and the extension of supply of electrical power. Hoisting equipment can be rented at reasonable rates from dealers who employ skilled maintenance men to care for the equipment.

Forms for Fire-Proofing.—Column forms may be built either from 1 or 2-in. lumber. Where it is desired to re-use the forms a number of times it will be found economical to use 2-in. boards. All forms should be painted or sprayed with oil. Steps should be taken to assure accurate fitting of the sections when they are set in place. The inside surfaces should be dressed true and where the columns are to be exposed triangular strips of wood or leather should be inserted at the corners. Column forms are usually fastened with metal column clamps which may be secured on a reasonable rental basis.

House Derricks.—Small house or breast derricks are principally used for setting stone. Usually, they are hand-operated, braking being accomplished by means of a "bull tail", consisting of a length of rope wrapped around the upper of the two shafts. These derricks will be found useful on every building job, but would be considerably safer if they had mechanical brakes.

Scaffolds.—The suspended type of scaffold is generally used for work on the outside of steel-frame buildings. It consists of a platform hung by wire cables from overhead supports. In general, there are two types of machines used for suspended scaffolds. The platform type has the winding drums and attachments secured to the platform and winds up the cables from the lower end by levers or cranks. The other, the over-head type, has the drums and attachments mounted on the outriggers at the top of the building. These are operated by endless ropes running over pulleys connected with the drums and hanging down to the lowest level at which the platform is to be used.

The cables for scaffolding support should be not less than ½ in. in diameter. Desirable safety expedients for the scaffolding platform include a head covering of boards and a side covering of wire mesh about 3 ft. high.

Electric Motors.—Electric motors should be operated strictly according to directions issued by the manufacturers. Controller handles should be advanced slowly to avoid burning out of coils and severe mechanical strains on the hoisting mechanism. A fuse or circuit breaker should be provided and adjusted to act at not more than 50% over-load. Solenoid brakes are desirable and form an additional safeguard. Fuses of the enclosed type should be installed whenever possible. Switchboards should be railed off.

A. J. HAMMOND, *Chairman,*
W. J. DEAN,
L. C. DILKS,
Committee.

CONSTRUCTION PLANT AND METHODS FOR CONCRETE BRIDGES

REPORT OF COMMITTEE

In attacking his problem, as to plant and methods, the concrete bridge builder must consider:*

(a) The character and quantity of units of work involved.
(b) The time allowed to do the work.
(c) The characteristics of the site.
(d) The history of the stream and the meteorological records.
(e) The supervisory and executive staff he has available for the work in hand.
(f) The available labor and mechanics.
(g) The sources of supply of materials and the probable continuity or dispatch with which they may be secured.
(h) The safety of men.

Inasmuch as plant and methods vary widely, depending on the character of foundations, this study will be developed under the following outline:†

1.—Concrete viaducts (with little or no water problem).
2.—Concrete bridges, having shallow foundations to rock or foundations resting on pile underpinning.
3.—Concrete bridges, having deep foundations where work under air is required.

(1) *The Method of Attack.*—This is perhaps the first thing to be settled. The bridge builder may begin work at either end of his structure, or in the middle, or combine these alternatives. The characteristics of the site will probably dictate whether he will start at the point nearest his plant and material storage yards, complete that to a certain stage, and then work over it to continue his operation of completing the remainder of the structure; or whether he will begin at the most remote point and work toward his plant. The difficulty, or lack of difficulty, of constructing and maintaining a temporary trestle, or crossing, might influence that decision.

For bridges over water the necessity of maintaining an open channel for navigation is a factor in arriving at the method of attack. A turbulent stream, or a fluctuating water level, might dictate the advisability of doing certain parts of the work in a favorable season of the year, and carrying on other parts, less affected by seasonal changes, when opportunity permits.

The method of attack must necessarily be based on some reasonable assumption as to stream performance. The stream may not perform as was expected from scrutinizing a 20-year history; the weather conditions may cut down or extend the length of the building season; the foundation shown on plans to rest on rock at Elevation 710 may have to be sunk to Elevation 685, requir-

* It is proposed, in the final report, to deal in detail with certain of these influences.

† These divisions overlap somewhat, and as a single bridge might comprehend all three divisions, this idea may be abandoned in compiling the final report. It is proposed that a plant layout plan typical of each of these types of bridges will accompany the report.

ing a change in method and loss of time—any of these will probably happen, and many others may happen. So the method of attack will be influenced accordingly.

Although plans made as to the order in which the work will be carried out may suffer reversal, and although parts of the work may require more time, or less time, than anticipated, yet it is advisable to prepare in advance of starting work a "progress schedule".

(2) *A Progress Schedule.**—Such a schedule is the result of dividing the work to be done by the time allowed. Plant and method must be made to harmonize with the progress schedule, and the wise builder will leave some margin for what he cannot control. Units of work in different sections should be shown on the progress schedule, and beside them should be set down the length of time required for each such operation. It is well to plat the whole performance on a chart, with time as ordinates, and items of work as abscissas. In this way, interference by seasonal floods, or cold weather, becomes easily apparent.

In all probability the original expectation of performance will have to be modified due to causes enumerated, or other causes of which no one has ever heard. If the departure from expectation is a material one, it becomes advisable to take stock of what is done to date, what remains to be done, and the time remaining in which to do the work; and then to make a new progress schedule. A progress schedule should be a guide for that part of the bridge builder's organization which buys materials and arranges delivery.

(3) *Choice of Equipment.*—Generally, the size of equipment, or the number of equipment units, will develop from a consideration of the practical average output per hour, of the hours per day worked, of the number of days allowed, and of the amount of work to be done. Builders are much more nearly in accord as to the amount of plant than as to the type most suitable to secure progress and economy.

Comparative plant layouts should be studied by considering the cost of use as set down in the "Plant Rental Schedule of the Associated General Contractors of America". A layout capable of wonderful performance might be economically unsound, when the cost of its use is considered either with respect to the units of work involved, or some less expensive, but adequate plant.

The Committee on Concrete Bridges, to develop the scope of its work, sets down in this preliminary report a general outline of items which now appear pertinent in a discussion of plant and methods for concrete bridges. Criticism and advice would be welcomed to indicate whether the proposed treatment of the subject will prove interesting, and to what extent it is desirable to go into detail. The Committee would also appreciate comment on whether the outline indicates that a report developed from it would overlap the work of other committees, or whether important items have been omitted from the outline.

(4) *Use of Equipment.*—The following outline covers subjects to be studied in terms of use and arrangement of equipment.

* The final report will contain a typical progress schedule.

A.—Development of Site for Work.—

Easements and right of way.
Roads.
Tracks.
Water supply.
Power:
  Electric motors and power lines.
  Steam.
  Gas.
  Diesel.
Light and wiring.
Compressed air.

Temporary Buildings:
  Offices.
  Tool rooms.
  Shop.
  Saw-mill.
  Lunch shelter.
  Camps.
Transportation of men
Storage space, for:
  Lumber.
  Timbers.
  Piling.
  Steel.
  Miscellaneous supplies.
  Concrete materials.
Unloading equipment.

B.—Viaducts.—The typical operations for which plant is required for viaducts are as follows:

Excavation:
  Earth, dry.
  Earth, wet.
  Rock, solid—dry.
  Rock, solid—wet.
  Rock, loose.

Pile-Driving:
  Of wood piles.
  Of concrete piles.
  With swing drivers.
  With skid drivers.
  With drop-hammers.
  With steam-hammers.
  With jetting-hammers.

Concrete Mixing:
  Aggregate storage and reclamation.
  Cement storage.
  Mixer bins.
  Batching:
    (1) Volume.
    (2) Weight.
  Water measuring.
  Mixers.

Pumping:
  (The influence of "pay lines", shoring, and the several kinds of excavating machinery available will be discussed in more detail in the final report.)

Shoring (But not Including Forms):
  Steel Centering:
    Trusses.
    Arches.
  Wood Centering:
    (Should be discussed as to selection of type for specific purposes.)

Transportation to Place:
  Towers and chutes.
  Locomotives and cars.
  Cableways.
  Carts.
Placing:
  Bottom dump buckets.
  Buggies.
  Tremies.
  Spading.
  Movable hoist towers.

It would appear that the consideration of concrete plant for bridges would lead to a discussion of such things as are required for production in large volume and continuously, this as differentiated from plant for concrete buildings.

C.—Concrete Bridges Having Shallow Sub-Aqueous Foundations.—The typical operations for which plant is required in this case are:

Excavation:
  Dredging.
  Clamshells.
  Shovels.
  Draglines.

Coffer-Dams:
  Earth dike.
  Crib.
  Wood-sheeting.
  Steel-piles.
  Pumping.

Floating Equipment:
  Barges.
  Derrick scows.
  Pile-driver scows.
  Trestles.

D.—Concrete Bridges Having Deep Foundations, Where Work Is Under Air.—The typical operations for which plant is required in such bridges are:

Air-locks.
Air supply.
Cutting-edges.
Caisson shafts:
- Wood.
- Steel.
- Concrete.

Sinking Methods.
Excavation:
- Buckets.
- Hoists.

Placing of Concrete:
- Plant and methods.

Several items that should be expanded and logically incorporated in the final report are: (1) Safety measures; (2) legal regulation of conduct of work; and (3) storage and rehabilitation of plant.

DONALD B. FEGLES, *Chairman.*
E. H. CONNOR,
Committee.

CITY PLANNING DIVISION

JANUARY 17, 1929—2:30 TO 5:00 P. M.

This meeting of the City Planning Division was the first to be held without the genial presence of its Secretary, the late Charles B. Ball, M. Am. Soc. C. E. To those who were familiar with his efficient direction of the Division, it seemed wholly fitting that at the suggestion of Chairman C. E. Grunsky the meeting should at the start stand in silent honor to Mr. Ball.

RELATION OF THE LAWYER TO THE OTHER PROFESSIONS ENGAGED IN CITY PLANNING

By Alfred Bettman,* Esq.

What Is Law?—At the outset, Mr. Bettman disclaimed any intent to resort to commonplace and platitudinous thoughts about the great friendship that should exist between all the learned professions. Instead his remarks were to be informal, albeit well justified by experience after considerable contact with engineers in city planning work. In effect, he would try to re-orient the lawyer, to "bring him back in some of the relationships in which he might be more useful than the other professions seemed to think."

"What is the nature of a law?" asked Mr. Bettman:

"A law is a rule of conduct laid down by some law-making organ to govern conduct of people and to be enforced by the Courts. It is not a rule that is drawn out of the air, it is not a piece of abstraction; it represents, or if it is to be effective it must represent, some consciousness of a need that has to be supplied through law, or some consciousness of an evil in existence that has to be removed through law."

Limitations of the Lawyer.—Obviously, he insisted, of the professions the business of which it is to ascertain the needs of city planning, the lawyer is not one. Experts are needed in physical matters—traffic and the size of streets—whereas a lawyer at best is only an expert in relation to law. Or in the field of city planning—providing open spaces and other necessities for health—this is the function of an expert, possibly in building construction or in sanitation, but certainly not of a lawyer.

Once the crying needs are determined, the removal of the evil likewise lies in some other province than the law, Mr. Bettman continued,

"Up to the point of the statement of the need, or of the evil, and the statement of the solution for the removal of the evil, there is no need—I speak broadly, of course—to draw in the lawyer. As a matter of fact, in the city planning movement in this country he has been constantly drawn in previous

* Attorney-at-Law, Cincinnati, Ohio.

to the engineering phase of the problem. Before the engineer or the public health man, or the economist has found out whether there is any engineering or economic need, or what it is, he has already asked the lawyer 'May we widen the streets?'

"Law Finds the Way.—When I was City Attorney of Cincinnati, the City Engineer and the Mayor were constantly asking 'If we should want to do so and so, may we do it?' My invariable answer was (after a few days' experience), 'If you ask the lawyer may we do it, he has to play safe and say "no" as a rule, but if you first find out what you want to do and then turn it over to the lawyer to tell you how to do it, so that his constructive talent is called into play, he will usually find a way.'

* * * * * *

"The premature injection of the lawyer, or the question 'May we do it?' causes what I would call an unscientific solution of things, a premature moulding of things to a supposed legal requirement, instead of a thorough-going and conscientious first working out of the data and the solution in terms of the profession of the man who is working them out, be he engineer, public health expert, or economist.

"Having worked out a thorough and fair plan, it is time to draw the lawyer in when you come to write the ordinances or write the laws."

Lawyer, Not Engineer, to Draw Ordinances.—Applying the same principles in their reverse directions, Mr. Bettman felt that engineers should likewise concede to the lawyer the task of formulating the statement of the need into language which can operate as law. Many laws, he feels, show this deficiency and, accordingly, suffer through imperfect phraseology, usage of technical words of legislation, and usage of the particular legislative terminology which traditionally is understood in that State, for example. A law cannot be transposed from New York to Ohio, or *vice versa,* without detriment; its terminology may prove strange to the new judges and officials and cause very contradictory interpretations.

Because of the peculiarity of the legal habits of the United States and its Constitutional law, whatever the legal zoning plan or subdivision regulation, sooner or later a Constitutional question will arise and will have to be met either within or without the Court room. The resulting relationship between engineers, economists, public health experts, and the lawyer, according to Mr. Bettman, is profound and of primary importance.

Reasonableness.—For example, frequently some phase of regulation, usually a physical phase affecting an individual property owner, is accused of being unconstitutional because it is "unreasonable". Presumably it means something "moderate, just sort of decent, in the middle of the road, something that will not cause the Courts to prick up their ears and their eyebrows."

"It has, however, a much more profound meaning than that. It means that the law is the result of the process of human reasoning and distinguished from the result of bias or corruption or guess work or emotionalism. When you say that a zone plan is reasonable, you do not mean that it is pleasant or nice; you mean that it is the result of the exercise of the human reason."

Law Reflects Expert Reasoning.—This, of course, introduces all the physical concepts of engineering and economics and, obviously, requires a thorough-going survey of the situation and factors as a preliminary to the drafting of

the law; that is, not only should it be the product of reasoning, but of expert reasoning. Whatever the general field of city planning legislation involves, said Mr. Bettman,

"In so far as the work of the other professions in the law is thoroughly and conscientiously done, in that far will the chances for the lawyer to maintain the constitutionality be increased, for in the long run law must respond to needs. One cannot have a system of law that will stand if it attempts to stand in the way of the necessities of the health, welfare, morals, or the convenience and safety of the people. It will break down by attempting to withstand it. The very purpose of law is to promote the health of the people and their convenience and their prosperity, and their order and their safety, and, therefore, any system of law that attempts to stand in the way of those things necessarily will fall down.

* * * * * * *

"*Formulating Work of Technicians.*—So it is the work of other professions, in the last analysis, to a far greater extent than the work of the lawyer. The lawyer is needed to present the work of the other professions to the legislatures, to the councils, to the people, to the Courts. He is supposed to have a gift for talking, but it is upon the consciousness and thoroughness of the work of the other professions that in the last analysis all legislation which in any way is related to the subject matter of our conference in city planning —all legislation depends for its constitutionality, for its validity, for its effectiveness, for its capacity to stand in practice and stand in law.

"In a general way, it is the work of the other professions to supply the scientific material, the scientific methods, the scientific work; then to call in the lawyer to formulate that work in terms of law and to sustain it, fortified by the consciousness and thoroughness and scientific spirit with which the work was done."

DISCUSSION

THE LAWYER AND CITY PLANNING

By Francis J. Mulvihill,* Assoc. M. Am. Soc. C. E.

Law Helps to "Accomplish".—In a prepared contribution, Francis J. Mulvihill, Assoc. M. Am. Soc. C. E., set out to stress the need for legal aid, advice, and assistance in city planning, and to cite examples illustrating the fact.

As Mr. Mulvihill expressed it:

"If you ask a youngster, member of the Boy Scouts of America, to explain the Scout motto, 'Be prepared,' he will, in effect, say, 'It means knowing a thing; what, where, when, and how to do it; then doing it.' Doing it—accomplishment—gives the lasting satisfaction. The lasting satisfactions from planning are truly in having plans executed, carried out, translated, so that they become accomplishment and achievement, existing in reality.

"It is apparent that the design, idea, is necessary before it can be accepted, sold (in the full salesmanship sense), or adopted. After that, it may be or may not be constructed. Consideration of various factors, physical, political,

* Chf., Div. of City Planning, and Municipal Engr., Bureau of Municipal Affairs, Harrisburg, Pa.

economic, and social, for example, if not adequately treated, or are slightingly treated, may each severally or collectively prevent accomplishment of the project in its finality. When each of the foregoing have been fully met the project appears to be doomed if the legal basis, all requirements of the law, has been overlooked."

Medical men, said Mr. Mulvihill, certainly fight disease and, likewise, professions engaged in city planning try to cure troubles; but they are even more interested in preventing them from occurring. Such, in short, is the function of the lawyer—as a preventive. Even the Government "City Planning Primer", prepared in the U. S. Department of Commerce, states that "planning rests on legal basis". Frequently, the Government officials of political subdivisions appeal for help to State authorities when as a matter of propriety they should consult their solicitors.

Zoning a Legal Matter.—In one particular phase of city planning—zoning —Mr. Mulvihill considered the lawyer especially needed. Without proper legal safeguards, any valuable accomplishment in zoning is well nigh impossible. He especially cited instances in New Jersey. As indicative of the attitude in his own work in Pennsylvania he stated:

"No zoning ordinance can be made proof against legal attack, but if proper precautions are taken in its preparation the machinery of administration will work more smoothly and liability to successful legal attack will be reduced to a minimum. The cost of the assistance and advice of competent consultants will probably be very amply compensated for in economy of administration and the forestalling of possible legal conflicts."

For the benefit of others interested in the subject, he referred to a number of books and periodicals dealing with this particular question.

WESTCHESTER COUNTY PLANNING AND PARK SYSTEM

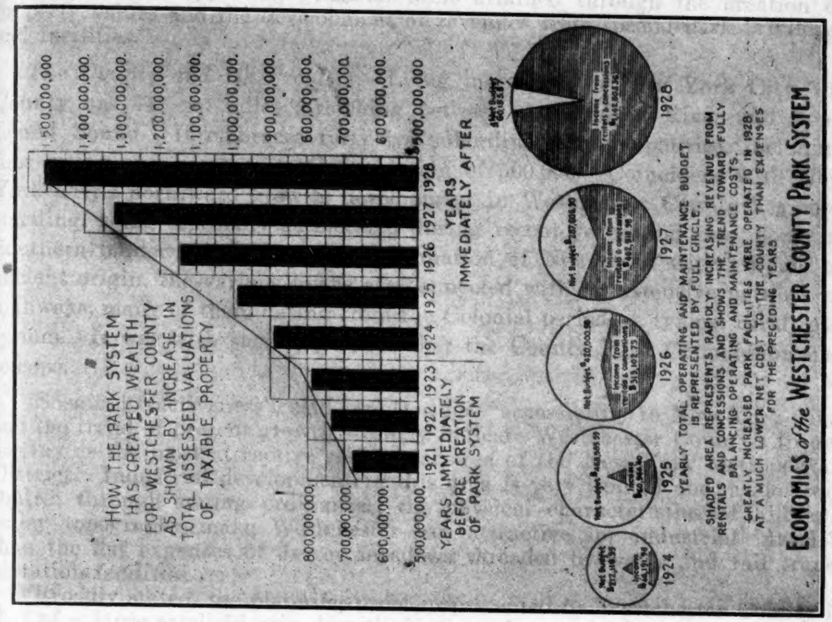
By Jay Downer,* M. Am. Soc. C. E.

Outstanding Accomplishment.—From his intimate experience as Chief Engineer of the Westchester County (New York), Park Commission, Jay Downer, M. Am. Soc. C. E., gave a lively account of the development and success of this work with numerous illustrations and diagrams. As a sort of a text he affirmed "Westchester County's experience has added to the evidence we have had before that good planning, well executed, is a great economic asset."

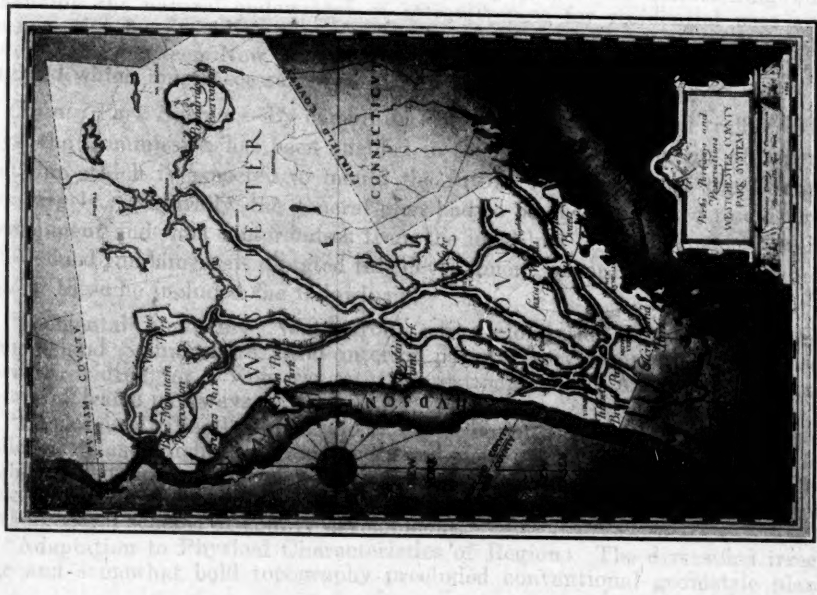
He made the startling claim that:

"Westchester County, New York, during the past five years has made a contribution to park history, regional planning, and practical execution hitherto unmatched by any other municipality in the world in an equal period of time. Three outstanding features may be offered in support of this rather broad claim: First, the comprehensiveness of the County Park System which includes water-front and interior recreational areas and reservations, and an extensive system of interconnected traffic parkways; second, the speed of exe-

* Chf. Engr., Westchester County Park Comm., Bronxville, N. Y.



FINANCING OF WESTCHESTER PARK SYSTEM GRAPHICALLY SHOWN.



PICTORIAL OUTLINE OF WESTCHESTER COUNTY PARK SYSTEM.

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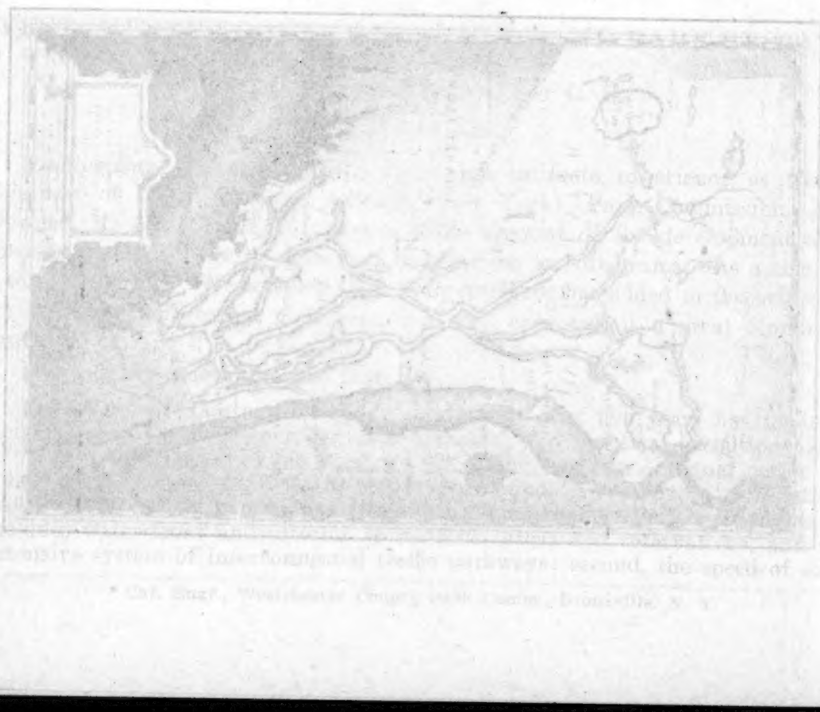
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cution in the acquisition of lands, development, and construction work; and, finally, the self-supporting financial basis attained through the creation of property values and the development of revenues from special park privileges and facilities."

The County and Its Problem.—Lying just north of New York City, the County has 448 sq. miles stretching between the Hudson River and Long Island Sound. It embraces forty-five subordinate municipalities, including four cities, and boasts a total population of 500 000. Coincident with New York City's northward growth, development in Westchester County has been startling, being especially accelerated during recent years, and now along its Southern border being almost a continuation of New York City. Being of ancient origin, the various towns are connected with an extensive network of highways, many of them dating from the Colonial period—a typical rural road system. It was with such a problem that the County Park Commission had to cope.

"Scenic attractiveness", said Mr. Downer, "accessibility to New York City, and the trend of present growth strongly indicate Westchester County's future destiny as the most attractive residential area of the New York Metropolitan District. Industrial development in this area is very limited and, aside from control through zoning ordinances, the physical characteristics of hill and valley topography make Westchester less attractive for industrial invasion than the flat expanses of Jersey meadows threaded by water and rail transportation facilities.

"Broadly stated, the planning problem presented by Westchester County is that of a large satelloid area directly in the only mainland path of growth of New York City and destined to be devoted to residential purposes as its major utilization, but with local distribution and retail business districts to supply the needs of a greatly increased future population. Among the leading essentials for the planning of this area is the requirement of maintaining and enhancing the natural endowment of attractiveness for residential uses and the necessity for providing open traffic ways for the large volume of through-motor traffic between New York City and Upper New York State and New England which must necessarily pass through Westchester County."

County Park System.—By combining county parks for arterial traffic parkways, the Commission has been enabled to form the main elements of a general plan which is expected to mould the future development of the County very largely. Naturally, the general plan had to be accommodated to existing development and thus compromises from the ideal had to be effected. Certain well-defined fundamentals dictated this development, according to Mr. Downer. Among these he included the following:

"Elemental Features: Water-front parks along the Hudson River and Long Island Sound shores, and interior parks, reservations, and forest preserves providing for a wide range of recreational uses. An interconnecting system of traffic parkways.

"Economic Utilization of Land: Selection of the cheapest undeveloped lands, such as marshes, stream, and pond margins, or rocky uplands undesirable and difficult to develop for residential or business uses, but ideal for park purposes and as such allocated to their most valuable economic utilization in the general scheme of county development.

"Adaptation to Physical Characteristics of Region: The diversified irregular and somewhat bold topography precluded conventional geometric plan-

ning. Such planning could not be rationally or economically adjusted to the physical conditions.

"Adjustment to Existing City, Town, and Village Developments: Interwoven with the foregoing was the necessity for effecting the most feasible compromises with existing conditions of the unplanned suburban growth.

"Rapidity of Execution: One of the most important elements was prompt action in acquiring lands in advance of rapidly rising values, leaving the shortest possible gap between planning and the execution of this phase of the program."

Parkways.—After showing the general layout of these parks and parkways, Mr. Downer continued:

"The north and south parkways, traversing the stream valleys in the direction of greatest traffic flow, serve the multiple purposes of open recreational areas, motor traffic routes of least resistance, the natural drainage routes of trunk sewers, and possible future locations of water supply conduits and other public utilities. The parkway reservations which include controlling strips of land on both sides of the streams also forestall the development of nuisance conditions such as developed along the Bronx River Parkway and led to its construction principally as a sanitary improvement project prior to the general county park program. The parkway system is co-ordinated with the older existing county highways."

The first of these parkways, Mr. Downer explained, called the Bronx River Parkway, was a joint undertaking between Westchester County and the City of New York. It extends 12 miles through the county and 3 miles within the city. About three years before this work was finished, the County adopted its expanded park program, working under a park commission of six members serving without compensation. One year was devoted to study, after which definite recommendations were made, all of which were adopted by the County Board of Supervisors and the requisite appropriations unanimously voted.

Extent of Park Development.—As a result of five years' active work, said Mr. Downer,

"The Commission has established a regional park system embracing within the limits of Westchester County, 16 671 acres of land, 140 miles of parkway routes, and 9 miles of beaches and shore lines along the Hudson River and Long Island Sound. This work has involved the acquisition of about 3 500 separate parcels of land, a large proportion of which was acquired by direct purchase from owners, thus avoiding the delays incidental to tedious condemnation proceedings.

"Coincidently with the acquisition of land, a construction and development program has carried to completion seven new large-scale recreational developments, including bathing beaches, bath houses, concrete swimming pools, a wide range of athletic, recreation, and picnicking grounds, three public golf courses, forest preserve areas, and a seaside amusement park embodying a unique and unusual feature under municipal operation. Under this development program 17 miles of fully developed traffic parkways have been completed giving a total of 29 miles, including the Bronx River Parkway."

Basic Practice in Parkway Design.—What Mr. Downer styled the "Westchester County type of parkway" is being recognized as the most economic solution. By establishing attractive residential zones along the roadways, valuations are greatly enhanced; this is largely in contrast with the depression of land values along trunk highways flanked by billboards. The West-

chester practice is to utilize the minimum width of 200 ft., enlarging this where cheap land is available or where water-surface development is feasible. Mr. Downer continued:

"As to the motor driveways, the standard practice is to lay pavements 40 ft. wide for four lines of traffic. Grade separations have been effected at all main intersecting thoroughfares which are carried over or under the parkway drive by bridges. The rigid-frame type of stone-faced reinforced concrete bridge has been applied to the grade-separation problems with very satisfactory results both as to economy of construction and pleasing architectural adaptations. On the Bronx River Parkway the type of pavement was a concrete base with $2\frac{1}{2}$-in. bituminous wearing surface, but the newer projects are standardized on reinforced concrete pavement, with asphaltic emulsion applied to eliminate glare. Grading and bridge openings are being carried through to a width of 60 ft., as the necessity for increasing the initial 40-ft. pavement width to 60 ft. in the not distant future is foreseen.

"*Economics.*—The park program is financed as to capital investment in lands and permanent improvements, through the standard type of County bonds running for 45 to 50 years and usually underwritten at an interest rate of 4 per cent. The yearly operation and maintenance charges are carried as an item of the general County budget raised by current tax revenue.

"From 1922 to the end of 1928, the total capital appropriations were as follows:

"Acquisition of lands..............	$32 297 000
Improvement and construction.....	18 515 900
Total....................	$50 812 900"

"Although not susceptible of exact calculation, it may reasonably be held that the creation of enhanced property valuations have fully offset the capital appropriations, regardless of the public welfare benefits of the park system. The sharp upward trend in assessed valuations immediately after the establishment of the park system is shown by the following figures:

"Year.	Total Assessed Valuation of Taxable Property.	Increase
1921...........................	$676 103 963	....
1922...........................	733 007 069	$56 903 106
1923...........................	788 029 096	55 022 027
1924...........................	891 331 983	103 302 887
1925...........................	987 068 857	95 736 874
1926...........................	1 143 871 106	156 802 249
1927...........................	1 318 826 453	174 955 347
1928...........................	1 501 531 153	182 704 700

"The influence of the park program is apparent in the increased valuations in 1924 and subsequent years. From 1924, when this influence became evident, up to and including 1928, the total increase in assessed valuations was $710 797 357. At the previous rate of increase the total for the same 5-year period would have been about $300 000 000. An increase of about $410 000 000, or about eight times the total amount of the appropriations, is, therefore, reasonably attributable to the park program."

Self-Support.—One of the outstanding results, remarked Mr. Downer, has been the attainment of a "self-supporting maintenance basis", by virtue of rentals, concessions, etc., for the use of the recreational and amusement features of the park system. He then quoted the following statistics to "show the rapid

trend of earnings toward completely balancing the operation and maintenance budget:

"Year.	Park Operation and Maintenance Budget.	Income from Park System.	Net Cost of Operation.
1922	$25 000.00	$272.50	$24 724.50
1923	54 000.00	126.00	53 874.00
1924	258 309.33	46 191.98	212 116.35
1925	529 529.99	60 944.40	468 585.99
1926	733 103.33	313 102.75	420 000.58
1927	750 125.28	462 518.98	287 606.30
1928	1 202 958.93	1 142 803.36	60 155.57

"Although 29 miles of modern parkways and an extensive system of recreational projects were in operation in 1928, the net cost to the County was only $60 155. The parkways yield no direct income, but are of incalculable value in the saving of time and in convenience and safety for the traveling public.

"*An Early Success.*—Although the importance of a regional park system as an item of major importance in planning for the future growth of Westchester was clearly foreseen, considerable courage was required to launch it as the largest scale improvement program ever undertaken by the County. But officials and leading citizens had full confidence that the ultimate results would amply justify the enterprise. The results which are shown by the foregoing balance sheet, frankly, have been attained in a much shorter period of time than originally was anticipated.

"The demonstration afforded by Westchester that the idealism of a well-conceived planning program may rest on a sound economic basis is providing stimulus to many other municipalities both in this country and abroad."

Parkways vs. Trunk Highways.—As any one familiar with the Westchester work would readily realize, the parkways are one of the fundamental factors. Perhaps this work is as widely and favorably known for this one feature as for any other. Certainly, these parkways seem to be one of the basic ideas or backbones of the entire work. Mr. Downer explained the underlying purpose of this part of the work, as follows:

"We are building parkways in Westchester instead of great, new trunk highways. This has been a fact which has been stressed at city planning conferences during the last two years by some of our eminent, far-thinking city planners, which is now getting into the public consciousness; the fact that we do not want to spend all our money on wide State roads, that we cannot afford to build too many super-highways, which are simply wide slabs of concrete without protection of frontages. There has been a good deal in the papers about that during the last few years, and if you knew as I do how many people are now coming to our office to study that feature—the distinction between a parkway and a broad State highway—you would realize that that is something to reckon with.

"In other words, the people are beginning to demand protection of frontages on these expensive improvements, not only that the improvement itself may be enjoyable instead of a disfigured thoroughfare, disfigured by cheap stands and billboards and tawdry business development, but that the improvement may eventually pay for itself in its influence upon the zone which it serves.

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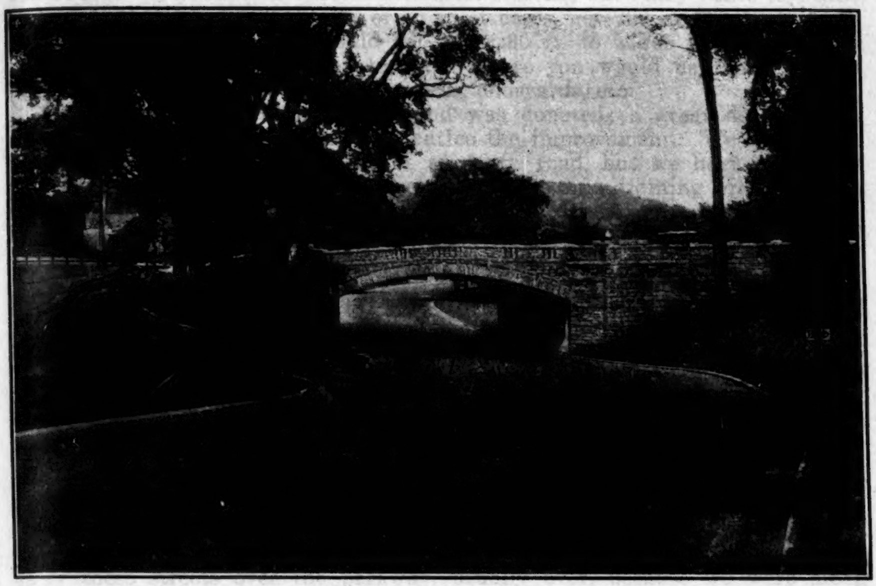
"*New Locations Cheapest.*—Another thing we have found in Westchester is that it seldom pays to lay out a great traffic route by widening an existing road. I am speaking now, of course, of suburban areas where you have a



WESTCHESTER COUNTY PARK SYSTEM: BRONX RIVER PARKWAY CROSSING ELIMINATION AT HARTSDALE, N. Y.

BRONX RIVER PARKWAY: TYPICAL VIEW NEAR FLEETWOOD, N. Y.

WESTCHESTER COUNTY PARK SYSTEM: BRONX RIVER PARKWAY CROSSING ELIMINATION AT HARTSDALE, N. Y.

BRONX RIVER PARKWAY: TYPICAL VIEW NEAR FLEETWOOD, N. Y.

great deal of vacant land, only sparsely developed, which could be utilized for new routes. We found in Westchester after starting out many times to widen highways, that we could always, or in most cases, get 200 or 300 ft. in back somewhere cheaper than we could get 20 or 30 ft. to widen an old highway where the frontages were improved, and where you would pay for damages in cutting off fronts of fences, front gates, dooryards, etc.

"So we went in behind, where land was donated; a great deal sold at prices below the market in order to entice the improvement. We have there not only a wide parkway instead of an 80-ft. road, but we have controlled frontages, we have a great deal of restriction on the adjoining property, and we have controlled, to a very large extent, access drives and crossings so that these can be developed as fast trafficways with the greatest margin of safety."

Planning Ahead of Buying.—Fortunately, the Commission had an excellent sample of park development in the Bronx River Parkway. This fired the imagination of the public so that they could visualize the future development. Hence,

"With that great start as an asset we bought as we planned, planning, of course, always ahead of our buying, and being sure that whatever we did was a part of a sound, general plan.

* * * * * * *

"The elimination of grade crossings was a moot question in 1916 and 1917 when we were planning this parkway, because it was very debatable. It seemed to us at that time if it was advisable to spend the amount of money required to carry these streets over the parkway, usually over, because we were along the river and the valley, it was easy to work out the grades and carry them over. We felt, however, that the automobile traffic which was then about one-twentieth of what it is now, would increase. Therefore, we decided to separate these grades. We omitted a couple of the worst ones because we did not have the courage to ask for the money. We have since gone back and done them."

DISCUSSION

PARK REVENUES

In Amusement Business.—Speaking of the attitude of the County toward the park work, S. D. Gridley, Esq., stated that there was no opposition to the splendid plans for parks and parkways even among the opponents of the present County policies. Both the engineering and the financial phases were highly approved; however, there was some opposition to having the County go into the amusement business by the purchase and improvement of beaches, and the sales of concessions (as at "Playland", Rye, N. Y., on Long Island Sound).

This prompted James L. Davis, M. Am. Soc. C. E., to inquire further concerning the machinery for producing these revenues, their amount, etc. Mr. Downer gave as one element, the adoption of a "hard-boiled policy of no passes". Then, again, all the rentals of county-owned property is in accordance with real estate appraisals rather than at normal amounts; this alone brings in $200 000 per year. Following still another engineering principle, the concessions are sold to the highest responsible bidder. More than half the total income is from the beach called "Playland"; this is, "frankly, a big amusement business in which the County has a monopoly".

Other Efforts.—Commenting on the Westchester work, and commending it as well, G. H. Norton, M. Am. Soc. C. E., compared it with the attempts made in Erie County, around Buffalo, N. Y. Practically the same efforts are being made. Parks have been established in the outlying regions—wild country—and this seems to have been a tremendous success. For one thing, it helps traffic by taking pleasure seekers off the highways. To a small extent the County has let out some privileges to concessionaires.

Comparing the Westchester work with the efforts in California, Chairman Grunsky noted with pleasure the attempt of a large State to accomplish similar results. Of the 1 000 miles of California coast lines, only 35 miles is publicly owned. It is a source of regret, he stated, that land once given away, must now be bought back from private owners at great expense for public parks.

STATUS AND PROGRESS IN THE ART OF CITY PLANNING*

BY STEPHEN CHILD,† M. AM. SOC. C. E.

Art and Science of City Planning.—One of the active men in City Planning in England, Raymond Unwin, says of his profession:

"The kind of knowledge needed is extensive rather than intensive, for there must be maintained a degree of detachment from the details of the problem, if the city and the life of the city are to be seen fairly and seen whole. The town designer (the city planner) must prepare his imagination for this work by watching and thinking over the phases of city life; meditating on their comparative manifestation in many towns; entering sympathetically into the needs and limitations, musing all the time on visions of how work might be made more efficient and town life more pleasant."

The planning of a city, the re-arrangement of its mis-planned sections and the provision for its new outlying growth is an immensely complicated problem. As a matter of fact, too, it is a problem that in the case of the growing city is never completely solved, but continues unceasingly and increasingly with that growth.

Before proceeding to plan, in order to do so intelligently first of all there must be a genuinely comprehensive civic survey. Then would come, although not always in this order, the major street plan or official city plan, and, with this, zoning, parks and parkways, and many other details.

So complicated and so numerous are these details that no one man can solve them all, but some man or group of men should possess sufficient knowledge and particularly the necessary degree of detachment from these details to be able to see the vast problem clearly and see it as a whole. After the various investigations, sociologic, economic, archeologic, engineering—after all the facts of the comprehensive civic survey—have been collected, tabulated, and studied, there comes a time when, to quote Kipling, it is necessary "to make a magic".

At this point it becomes clear that the problem is artistic, because the same rules apply to the preparation of the plan of a convenient and workable city as to the creation of any artistic design. There must be the same careful proportioning of the different parts; there must be, for example, complete harmony of relation between the industrial, commercial, and residential sections of the city—a proper arrangement of civic and governmental centers, of industrial and recreational centers, of park and playground systems. All these features and many others must be linked by parkways, boulevards, major highways, and minor streets, into one harmonious whole. Imagination, vision, and the principles of design must be used to correlate all these features so as not to stifle or impede, but so as to guide and promote, inevitable future growth. This is what makes a city plan a unit, a whole, rather than a haphazard collection of unrelated things—this is the art of City Planning.

* Abstracted by C. E. Grunsky, Past-President, Am. Soc. C. E., and Chairman of the City Planning Division, from a complete report by Stephen Child, M. Am. Soc. C. E.

† Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

The Official City Plan.—With a comprehensive civic survey in hand, the commission, advisers, and consultants should prepare the major street plan, which thus becomes the official city plan. This will propose many projects, differing in every community—civic centers; street widenings; the extension of old and the laying out of new streets; park and parkway systems; and playgrounds. It will include plans for water, gas, telephone and power mains, transportation systems, grade-crossing elimination, and housing betterment. All these should be based on the data in regard to population expected to be served and its probable density as determined from the comprehensive civic survey.

The official city plan will also include a program or order of procedure under which these betterments may best be taken up, together with suggestions for financing them. This is one of the most important details of city planning.

Zoning.—According to the Advisory Committee on City Planning and Zoning of the U. S. Department of Commerce:

"Zoning is the application of common sense, good judgment, and fairness to the public regulations governing the use of private real estate. It is a painstaking, honest effort to provide each district, neighborhood, or zone, as far as may be possible, with just such protection as well as just such liberty as are sensible in that particular district or neighborhood, giving every one, home owner, shop keeper, or manufacturer, the opportunity for the reasonable enjoyment of his rights. At the same time it protects him from unreasonable injury by neighbors who would seek private gain at his expense."

Utilization of Land and the Control of Sub-Divisions.—During recent years, intensive and invaluable studies have been made in regard to the basic principles underlying the utilization of lands. These studies are still in the making and no student of the subjects of city and regional planning should fail to become familiar with them.

They will give all city planners an immense amount of helpful data. For the present, however, one of the most authoritative sources of information in regard to the use of land and particularly the control of sub-divisions, is to be found in the publications of the Advisory Committee on City Planning and Zoning of the U. S. Department of Commerce, particularly the Standard City Planning Enabling Act prepared by this Committee.

Continuity of City Planning.—All who are well informed about city planning are agreed that it is a never ending job—that cities cease to need to plan only when they cease to grow, that every community that hopes to achieve physical as well as economical results from its efforts must realize that this continuity of endeavor must follow a carefully prepared program. The day is now past when city planning service can be completely satisfied by a two or three weeks' visit of an expert with intensive but necessarily hurried study followed by a report. Such documents are usually soon laid aside and forgotten.

Continuous city planning means correlating all features into a city planning policy or program to be as definite as circumstances will permit. Such a program must control the city's growth and direct it along lines that are most appropriate.

Regional Planning.—To most cases the arbitrarily established municipal boundaries seldom coincide with comprehensive community needs—that to solve many of the features of the planning of a community it is imperative to go beyond its legal limits. Hence, regional planning has become a necessity.

The determining of the limits of a region is a very difficult problem. Many factors must be considered—drainage, sewage disposal, water supply and conservation, transportation, etc. A region should be so delimited that it will include all genuine regional community interests.

Having done this intelligently, regional planning may well follow practically the same steps as have been pointed out for the city.

POWER DIVISION MEETING

JANUARY 17, 1929, 2:30 TO 5:00 P. M.

The Power Division, in discussing ice problems at hydro-electric plants and the underlying principles of pumped-storage hydro-electric plants, touched on two subjects of interest to hydraulic engineers. Although no report of the Division Committee on Ice Problems was presented, owing to the absence of its Chairman, F. C. Shenehon, M. Am. Soc. C. E., the meeting was fortunate in hearing Dr. Howard T. Barnes, of McGill University, deliver a paper on the "Constitution of Water and Nature of Ice". Dr. Barnes outlined a theory, new probably to many, on the formation of ice and gave results of experiments to determine its nature.

CONSTITUTION OF WATER AND NATURE OF ICE

BY HOWARD T. BARNES,* ESQ.

***Kinds of Water.*—For many years, Dr. Barnes stated, physicists and chemists have held the theory that water is a complex liquid, that it contains ice at practically all temperatures. Quoting from his paper, in explanation of this theory, he stated that,**

"Sutherland suggested that the properties of water could be interpreted by assuming that the pure substance known to the chemists as H_2O, could exist only in the form of dry steam or vapor; that liquid water was a double molecule or $(H_2O)_2$, and that ice was a triple molecule $(H_2O)_3$ which forms at once as an associative product, giving a liquid mixture of $(H_2O)_3$ and $(H_2O)_2$ in proportions which depend on the temperature. As the chemist uses H_2O very freely as a symbol of water, or hydrogen oxide, for international convenience Sutherland gave names to these associated molecules. Thus, the simple molecule was called hydrol, the double molecule, dihydrol and the triple molecule, trihydrol. Steam is, therefore, pure hydrol, ice is pure trihydrol, and water is a mixture of dihydrol and trihydrol."

Bearing on the theory of the constitution of water and nature of ice, he presented salient points including experimental findings. As evidence of the complex nature of water, he mentioned studies of the density, optical refraction, infra-red absorption, surface tension, surface films, etc.

***Density.*—Above 4° cent., the point of maximum density or minimum volume, stated Dr. Barnes, the ordinary laws of expansion apply; whereas below 4° cent., an anomaly exists. The curve represents:**

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* Prof., McGill Univ., Toronto, Ont., Canada.

"A resultant of two physical effects characteristic of two liquids: (1) The pure ingredient, water; and (2), a second ingredient which must be regarded as liquid ice. The complete curve of change of density with temperature can be obtained in a simple formula, by assuming that liquid water is altered by adding portions of the ingredient to which the normal density of ice is attached.

"This mixture changes as the temperature changes. Commencing with water at 0° cent., and increasing the temperature, the amount of liquid ice grows less causing a shrinkage of the co-volume, which masks the normal expansion of the liquid water. At 4° cent. there is an exact balance between the shrinkage due to the melting ice and the expansion due to the liquid water and liquid ice. Above 4° the expansion of the water mixture grows greater as the proportion of liquid ice becomes less.

"Sutherland calculated the value of the volume of this liquid ice to be 1.0894 which is very nearly that of solid ice. Notwithstanding this considerable forcing apart of the molecules of water against their mutual attraction for which we should expect energy would have to be put into the water to solidify it, we find actually heat has to be taken out of the water to change it to ice. For comparison we may consider the expansion of metals when they melt. There is on an average of such solids a melting expansion of 3.3%, whereas with water there is a contraction of 8.3 per cent.

"Ice in its solid form has a density of 0.9166 and if it could meet and expand without dissociation like a metal by about 3 or 4%, the density of the liquid ice at 0° ought to be about 0.88. Sutherland calculates the varying proportions of one ingredient with the other resulting as follows:

0° cent.	37.5%
20° "	32.1%
40° "	28.4%
60° "	25.5%
80° "	23.4%
100° "	21.7%
198° "	16.5%

"At the critical temperature, which is about 368° cent., water must consist of nearly pure dihydrol, or, in other words, the ice in solution nearly disappears at the critical temperature. It seems evident, then, that the melting point of ice is not a true physical melting point but a temperature of dissociation unlike the physical melting point of metals."

Optical refraction measurements, studies of the absorption spectrum, and other findings on the physical properties of water are indicative of a complex liquid. Dr. Barnes made clear that, whatever may be the ultimate assigned to the ice molecule in solution, or to the pure water structure, it did not vitiate important evidence of a complex structure.

Colloidal Ice.—In considering water as a colloidal solution of ice, there is no microscopic evidence to show colloidal ice particles in water at temperatures above the freezing point. Dr. Barnes stated:

"Just at the freezing point the colloidal particles of ice are ready to form complex groups of sufficiently large dimensions to be distinguished in the microscope. When a fall of temperature takes place even the smallest fraction of a degree below the freezing point, the exceedingly viscous particles of colloidal ice rapidly agglomerate and pass to the true ice crystal."

Microscopic photographs of ice particles as they precipitate from the water show that they are disk-shaped and devoid of crystalline form. The particles, if undisturbed, will grow into a form similar to a snow crystal.

Frazil Ice.—Coming to a more definite engineering conception, he continued:

"In all running streams of open watercourses the most troublesome form of ice is known as 'frazil'; to the water-power operator it is exceedingly troublesome. With the first cold weather in the autumn when the water comes to the freezing point there is often a very large and sudden formation of frazil. The explanation of this is exceedingly simple when we realize that the entire body of the stream is nearly 40% colloidal ice before freezing and coagulation takes place when the temperature of the water has dropped a few thousandths of a degree. This mass of ice rapidly coagulates into streamers and subsequently into lumps and large clots which are carried in the current to great distances. So abundant is this formation that within a few minutes the whole stream may appear to be loaded with sand. During this time of supercooling these clotted masses of colloidal ice grow rapidly and freeze to any object with which they come in contact.

"So delicate is this balance of temperature which determines the sticking properties of this ice that a change of a thousandth of a degree is sufficient to prevent the formation or accelerate the growth of this ice. This delicate balancing of the forces of Nature is easily explained when we understand the true nature of the water structure and that the freezing point of water represents a chemical change rather than the physical one.

"During the processes of formation the streamers and curtains of frazil ice form throughout the whole body of the water resembling a subaqueous fog. It occurs with the same suddenness as a fog in the air and its characteristics very closely resemble it. It is dispelled almost instantaneously by the light of the sun and becomes very sensitive to small temperature changes in the water.

"The light of the sun produces a direct action on the ice particles as well as an indirect action by warming the water. There are two ways that the frazil fog can be dispelled: One is through this direct action of radiant energy destroying the agglomerating properties of these colloiding ice particles; and, second, there is the action of heat in warming the water, elevating the temperature, and thereby mechanically acting on the particles of ice by melting them."

Limited Ice-Forming Power.—The fact that the latent heat of formation of solid ice from liquid ice is only 16 calories explains why such a small amount of radiant energy loosens solidly frozen ice crystals and causes small particles of colloidal ice to disappear.

In order to prove some of the points in connection with the ice-forming power of water, Dr. Barnes described a number of experiments, using lantern slides to illustrate his results. In explanation of the fact that the first run of ice is always the heaviest and that continued cold weather results in less ice, an experiment was conducted with fresh water in a tank, with the following interesting results:

"After the expiration of a certain time the ice was extracted and measured and then fresh ice was allowed to form. In this experiment after the first half hour a pailful of slush was taken out of the tank. It required a full hour after that to produce another pailful of slush and two hours more were required to get a third of a pailful. After four hours more, little or no ice was produced although the cooling effect was the same. This indicates that water may be exhausted of its ice-forming power by repeated freezing, and that this ice-forming power can only be restored by allowing the water to stand for 24 hours or so, or by replacing the tank with water that has not been frozen.

"*Warming Water to Produce Ice.*—The passive state of water can be explained on the ground that the time required for restoration of the trihydrol in solution is finite, and this is, at the temperature of freezing, considerably slower than at a higher temperature. The ice-forming power was restored by warming the water to room temperature and then cooling, indicating that the equilibrium is established with greater rapidity at higher temperatures. We have, therefore, the interesting anomaly of producing ice in water by warming it.

"This little experiment throws considerable light on the ice-forming power in Nature for it is a well known fact that the first run of ice is always the heaviest and continued cold weather results in less ice trouble. Hitherto no explanation has been given for this fact but it is apparent that it is a real physical phenomenon. Alternating periods of mild and cold weather give greater ice trouble than long continued spells of cold."

Photographs were also shown to illustrate the ability of different surfaces to collect ice. It was found that a frayed rope or string is effective in attracting ice out of water, but that steel wool seemed to be the best. Brass, copper, and wrought iron are also good, whereas cast iron has been found to be inefficient.

Anchor Ice.—The effect of the light from ordinary lamps of various sizes in dissipating ice was also demonstrated, in order to explain the effects of sunlight. It is well known that anchor ice is one of the most troublesome formations in northern rivers. This ice, stated Dr. Barnes,

"Occurs in rapids and streams up to a depth of 30 ft., and depends upon the clearness of the water and the rapidity of flow. It is exceedingly sensitive to daylight, and such formations as occur at night are immediately relieved in their depth by daylight. This anchor ice is formed in rapids even when the temperature is considerably above the freezing point. It forms more slowly with a greater degree of cooling in deeper water.

"Correspondingly, the melting of this ice is more rapid in the shallower streams on the advent of daylight and during very cold weather the anchor ice may remain for several days on the bottom in very deep streams. Thus, we will have on a river a run of anchor ice in the morning from the shallower parts and in the afternoon from the deeper portions, establishing thereby a very valuable distribution of ice flow in an open stream, for the greater abundance of ice from the shallower portion has passed before the ice from the deeper portion rises to cover the surfaces.

"*Surface Ice.*—The formation of surface ice is a study in itself and cannot be treated here. Its growth is exceedingly slow after it has achieved a certain depth, and it is interesting to observe the formation of the crystals of this ice by the accumulation by layer after layer of ice disks on the under side. Very old surface ice gradually becomes coarser in structure owing to the fact that the large crystals consume the smaller ones. This is noticed also in old glacial snow accumulations. After many years of continual cold the glacier becomes coarse-grained and the snow much more granular in structure.

"The problem of ice formations in power developments can be more easily explained when the constitution of water is understood. It is hoped that the theories outlined here, together with the experimental verifications of these theories, may assist in the better understanding of power-house operation. That it has already given rise to helpful remedial measures is now well known and in the future we can look confidently for still greater and more powerful methods to be devised whereby ice can be destroyed in the running stream while it is still liquid and before it has a chance to coagulate into a form more difficult to handle."

DISCUSSION

ICE PROBLEMS

By William T. Walker,* M. Am. Soc. C. E.

In the absence of William T. Walker, M. Am. Soc. C. E., his statement was read by George A. Orrok, M. Am. Soc. C. E. Mr. Walker described how an accidental re-circulation of discharged condenser water served to warm the cold water at an intake sufficiently to dissipate needle ice without the use of live steam.

Use of Live Steam.—The original circulating-water intake of the Riverside Plant of the Minneapolis General Electric Company was in the main channel of the river about 100 ft. from the dock line, and was a submerged structure partly surrounded by a loose stone barrier which formed a horseshoe-shaped lagoon about 60 ft. wide and 100 ft. long with the open end about 75 ft. down stream from the intake. As Mr. Walker described conditions:

"The inlet was, therefore, in practically dead water and was quite effectually protected from floating material and frazil ice. Located a few feet ahead of the circulating pumps was a second set of racks used to catch any floating material which failed to lodge on the first set of submerged racks. On this second set of racks running from top to bottom was placed a series of perforated pipes connected to the main steam supply and when the occasion seemed to require, live steam was turned in. Since this original station was only 12 000 kw. capacity and under full load condition less than 25 000 gal. of water per min. were required and the plant capacity was needed for very short periods, the entire arrangement worked most satisfactorily.

"*Value of Circulating Water Discovered.*—It is not strange that when the first addition was made to the plant to more than double its capacity the same general arrangement of steam pipes was placed upon the racks of the new screen house. But the new conditions were far different from the old. The new screen house was located at the dock line and therefore not so well protected from floating material. The load had now increased so that a large portion of the total capacity was required from 16 to 20 hours each day and the boilers had to be operated at high ratings and the best possible efficiencies in order to carry the load. In the first case a few hundred pounds of steam could be discharged into the circulating water without materially effecting a very low over-all efficiency, but under the new condition it was discovered that to heat 80 000 gal. of water per min. from the cold to the warm side of 32°, required more steam than the plant could well spare."

In the operation of the plant, Mr. Walker explained, a timber boom had been used to divert floating material from the screen house; also as a walkway for operators to break up ice and log jams. Thus, it happened that this boom was moved down stream on one occasion so that it came directly in front of the discharge tunnel, in such a way as to divert part of the discharge circulating water back into the screen house. As Mr. Walker remarked:

"The result of this accidental re-circulation was to warm the water at the intake from 2 to 5°; obviously, all needle ice promptly disappeared without the use of live steam jets.

"*Economies Effected.*—The saving in fuel thus effected for a station of, say, 60 000 kw. capacity, is quite an item. Such a station, when operating

* Constr. Engr., Northern States Power Co., Minneapolis, Minn.

a circulating pump to capacity, would need about 225 cu. ft. per sec. of circulating water. In order to prevent the accumulation of frazil ice during periods when this ice is forming, it is necessary to raise the temperature of the water about 0.5° Fahr. This rise is not as great as when 're-circulating', but is regarded as sufficient to prevent the accumulation of ice on the screens.

"If, together with the above data, we assume that the station operates with a boiler-room efficiency of 80% and burns 13 000 B. t. u. coal, the following calculation will show the amount of coal required per hour to heat the circulating water:

$$\frac{100\,000 \times 8.33 \times 60 \times 0.5}{13\,000 \times 0.8} = 2\,400 \text{ lb. per hour}$$

With coal at $6 per ton, this amounts to $173 per day.

"This calculation does not consider the actual melting of the ice. For ordinary cases, the steam calculated to raise the circulating water temperature 0.5° Fahr. would take care of the ice, although, of course, the actual temperature rise of the circulating water would not be that much. With large percentages of frazil ice involved (as is the case when sudden drops in temperature occur before an ice sheet is formed over the river) the steam consumption would be very much greater.

"The lower the temperature of the circulating water in the condenser the higher will be the vacuum attained, and the higher the vacuum attained the greater will be the efficiency of the steam turbine. Here is a case where theory and practice are somewhat at variance, for while the ice-cold circulating water tends to produce a low vacuum it also absorbs so much of the heat from the condensate that the total result is an actual loss in efficiency."

* * * * * * * * *

"The recirculating of a part of the condensing water has therefore resulted in two decided benefits: (1) The effectual elimination of all danger from needle or frazil ice clogging the rack or condenser and this without the use of live steam; and (2), increasing the over-all efficiency by increasing the temperature of the condensate."

Housing Trash Racks.—With respect to hydro-electric plants, where the tops of the racks which are just ahead of the wheels are exposed to the weather serious trouble may be expected from frazil ice. The reason is that the ice needles readily adhere to the colder steel bars, which obstruction lowers the water back of the rack, causing further exposure of the steel bars and lowering the ice wall. In this connection, Mr. Walker states,

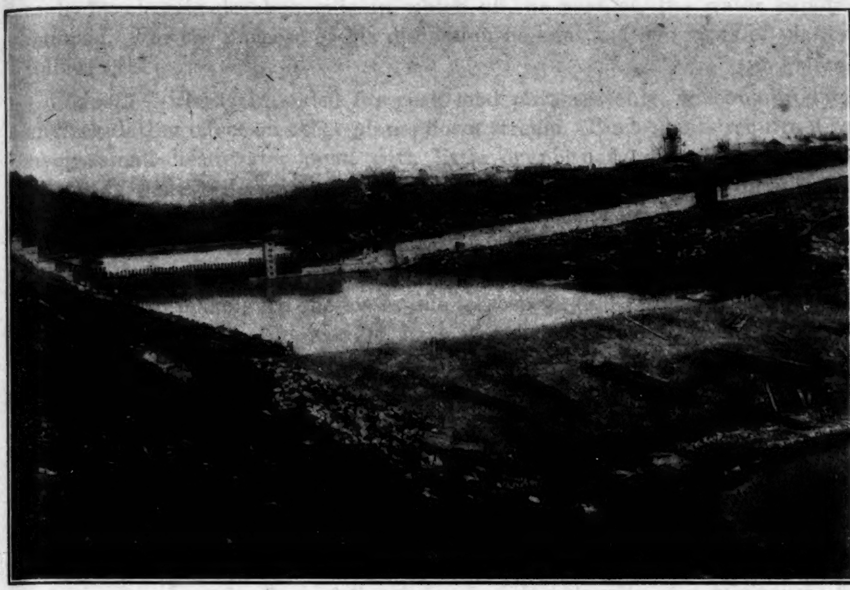
"We have completely solved our problems by housing over the racks and keeping the temperature within this house at 45 to 60° Fahr. So long as we are able to keep the ice needles passing through the racks we have little to worry about from that source for there is little possibility that enough ice will lodge in the scroll chamber to materially affect the operation of the units."

ROCKY RIVER HYDRO-ELECTRIC DEVELOPMENT

By Joel D. Justin,* M. Am. Soc. C. E.

In his paper, "The Rocky River Plant of the Connecticut Light and Power Company", Joel D. Justin, M. Am. Soc. C. E., first discussed the economics

* Hydr. Engr., U. G. I. Contr. Co., Philadelphia, Pa.



ROCKY RIVER MAIN DAM, SHOWING METHOD OF CONSTRUCTION.

ROCKY RIVER DEVELOPMENT, SHOWING POWER HOUSE, SUB-STATION, PENSTOCK, AND SURGE TANK.

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of hydro-electric developments in which all, or part of, the water supply is pumped. For the purpose of his discussion he analyzed two types of plants as follows:

Type 1.—Plants intended for peak load purposes only, without producing any regulating effect on other plants down stream. Such plants require a relatively small head-water pond with little or no natural inflow and also a relatively small tail-water pond. The water is shunted back and forth between these two ponds, which need be only of sufficient capacity for the operation of the plant on a weekly or daily cycle. This is essentially a dry hydro-electric plant. Only sufficient water supply is required for the initial filling of the ponds and to make good the losses due to seepage and evaporation.*

Type 2.—Plants intended to have a seasonal regulating effect on other plants lower down on the water-shed and also to serve as peak-load plants. The head-water of such a plant consists of a storage reservoir of considerable size. The natural inflow into this reservoir may be insignificant or it may furnish a material part of the water supply required. The tail-water is either a relatively large river, or the head-water of another hydro-electric development on that river. In this type of plant, the discharge goes through one or more other hydro-electric plants on the main river.

The power discharge from the plant, Mr. Justin explained, is a maximum at seasons when the flow of the main river is naturally at a minimum; thus, the plant operates to increase the firm power of the plants on the main river. In this respect, it operates much the same as any storage reservoir, the discharge of which is utilized for producing power. The essential difference is, that because of the deficient water supply of the small tributary, it is necessary to pump water during times of high flow in the main river up into the reservoir in order to fill it for use during times of low flow. Generally, the same conduit is used for pumping water into the reservoir and letting it down to the turbines. The Rocky River plant is of this type.

Economics of Type 1 Plant.—As it is usually economical to continue old steam plants for use over the peak, Mr. Justin believed that, in many cases in America, the Type 1 plant must be considered as a competitor of such plants for carrying the peak of the annual load. He states, however, that it will not usually be economical to supersede an antiquated steam plant with a Type 1 plant unless the total annual cost including fixed charges is less than the operating cost plus taxes and insurance for the steam plant.

Considering that a water supply is not essential and that all that is required is a site near the market where there are small basins for head-water and tail-water ponds, with a relatively large differential head, and a relatively short connecting conduit, it appears probable that there are a number of places in America where such "dry hydro plants" might prove economic.

General Economics of Type 2 Plant.—One function of Type 2 plants, Mr. Justin pointed out, was to regulate the output of a hydro-electric system, increasing the minimum flow of the main river and the primary energy output of the system and thus increase the capacity which may be installed as

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2459.

firm capacity on the load curve. The economic considerations leading to their adoption are very similar to those for a power plant with large storage reservoir, but with this difference—the operating cost of the Type 2 plant is higher by the cost of the pumping necessary to produce the regulating effect required. Manifestly, a Type 2 pumped-storage plant would not be built if an equally economical site with a sufficient water supply from natural run-off were available.

In the second place, these plants may perform the same function as the Type 1 plants, thus acting as peak-load plants as well as regulators. The power plant installation of a Type 2 plant may have to be based on the same grounds as a Type 1 plant, the remainder of the cost being justified by the seasonal energy regulating effect of the storage on this and other hydro-electric plants. A cost of only $120 per kw. may be justified for the peak-load plant, but the seasonal storage feature may justify a much greater capital expenditure, dependent on the head benefited, the installation in plants below, and the load curve.

Aim of Rocky River Development.—Following this general explanation, Mr. Justin proceeded to a detailed discussion of the Rocky River development, a Type 2 plant. Quoting from his paper he stated:

"Previous to the construction of the Rocky River Development, The Connecticut Light and Power Company, a subsidiary of the Connecticut Electric Service Company, had owned and operated two hydro-electric developments on the Housatonic River for many years; the Stevenson Development, located below the mouth of Rocky River, having an installed capacity of 18 750 kw., with a gross head of about 73 ft., and the Bulls Bridge Development located above the mouth of Rocky River, having an installed capacity of 7 200 kw., with a gross head of about 109 ft.

* * * * * * * * *

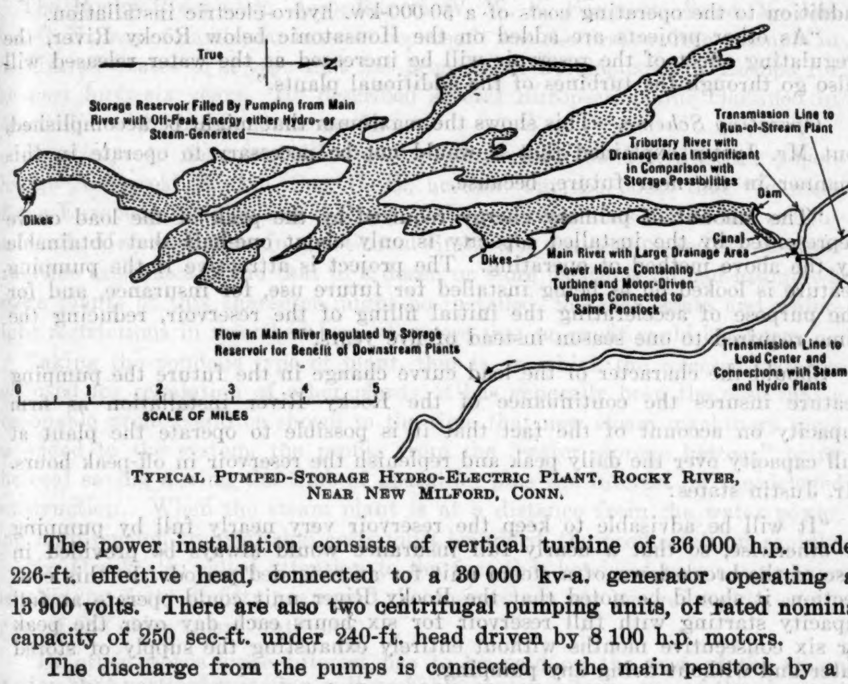
"Owing to the very low minimum flow in the river and the shape and size of the load curve, it had been found by experience that during a minimum flow period the two plants unregulated could be relied on for a firm capacity of only 6 700 kw.

"In order to make the capacity already installed (25 950 kw.) firm capacity which could be relied on to do the same work on the portion of the load curve assigned to it as steam capacity, some form of regulation was required."

Storage Basin.—The Rocky River Basin provided the means for such regulation and the dam sites permitted large storage of relatively low cost. The rapid descent from the dam to the main river also favored a low cost installation with a gross maximum head of 230 ft., which could be used as a regulator of the energy output of the remainder of the system and also as a peak-load plant.

The only drawback to the situation, Mr. Justin explained, was that the basin was so large in comparison with the tributary drainage area, that about five years would have been required to fill the reservoir. To overcome the difficulty two electrically driven pumps of large capacity were installed to discharge into the reservoir from the main river in times of high water. In a single season these pumps filled the reservoir which had a net capacity of 5 900 000 000 cu. ft., whereas the drainage area was only 40 sq. miles.

General Description of Power Development.—In addition to the Guarding Hill, or main dam, an earth structure about 100 ft. high and 1 000 ft. long, there are four other smaller dams. Two of these are of earth and two are concrete structures. The outlet canal from the main dam has a water depth of 40 ft., and is formed partly in deep hillside cut and partly in embankment. It is about 3 000 ft. long and delivers the water to a 15-ft. wood stave pipe, about 1 000 ft. long. From the surge bank at the end of the wood stave pipe, the water enters the penstock, which tapers from 13 ft. to 11 ft. in diameter, to the power house on the Housatonic River. The photograph shows these essential features.



TYPICAL PUMPED-STORAGE HYDRO-ELECTRIC PLANT, ROCKY RIVER, NEAR NEW MILFORD, CONN.

The power installation consists of vertical turbine of 36 000 h.p. under 226-ft. effective head, connected to a 30 000 kv-a. generator operating at 13 900 volts. There are also two centrifugal pumping units, of rated nominal capacity of 250 sec-ft. under 240-ft. head driven by 8 100 h.p. motors.

The discharge from the pumps is connected to the main penstock by a Y near the power house, so that the pumps may be used to discharge water into the reservoir whenever the generating unit is not in use.

Utilization of Rocky River.—In discussing the maximum utilization of the Rocky River plant, Mr. Justin stated that with the present hydro-electric installation on the main river (25 950 kw.) it would be possible to increase the 24-hour power from a minimum of 1 513 kw. to 11 450 kw.—an increase in firm 24-hour power, due to regulation, of 9 937 kw., or nearly 660 per cent. This increase would be accomplished by drawing on the Rocky River plant whenever the power supply in the main river was less than enough to produce 24-hour power of 11 450 kw.; whenever the supply is more than sufficient, the pumps instead of the generators in the Rocky River plant would operate in off-peak periods to pump surplus water into the reservoir, to be used to produce peak energy. Mr. Justin continued:

"Before regulation, it had been found that of the then existing installation of 25 950 kw. only 6 700 kw. was firm on the load curve at time of minimum flow. The minimum load factor, therefore, was $\frac{(1\,513)}{6\,700} = 22.6$ per cent.

"On the same load-factor basis after regulation the firm capacity would be $\frac{11\,450}{0.226} = 50\,700$ kw. (if the installation were available) which, of course, also represents an increase of about 660 per cent.

"As to the pumping, a large part would be done with surplus hydro-electric energy which would otherwise go to waste over the dams. If, however, the pumping were all done by off-peak steam-generated energy it would cost from $50 000 to $70 000 per year, which, for the results attained, is not a large addition to the operating costs of a 50 000-kw. hydro-electric installation.

"As other projects are added on the Housatonic below Rocky River, the regulating effect of the reservoir will be increased as the water released will also go through the turbines of the additional plants."

Operating Scheme.—This shows the maximum that might be accomplished, but Mr. Justin explained that it would not be necessary to operate in this manner in the near future, because,

"The amount of primary energy required by the peak of the load curve represented by the installed capacity is only about one-half that obtainable by the above method of operating. The project is attractive if the pumping feature is looked on as being installed for future use, for insurance, and for the purpose of accelerating the initial filling of the reservoir, reducing the time required to one season instead of five years."

Should the character of the load curve change in the future the pumping feature insures the continuance of the Rocky River installation as firm capacity on account of the fact that it is possible to operate the plant at full capacity over the daily peak and replenish the reservoir in off-peak hours. Mr. Justin states:

"It will be advisable to keep the reservoir very nearly full by pumping or otherwise, so that a nearly full insurance would always be provided in case of the break-down of a steam unit for an extended period. In this connection, it should be noted that the Rocky River unit could operate at full capacity starting with full reservoir for six hours each day over the peak for six consecutive months without entirely exhausting the supply of stored water and without doing any pumping.

"*Cheap Energy for Pumping Available.*—It has been found that off-peak hydro-electric energy can be obtained for pumping from the other plants of The Connecticut Light and Power Company and from plants of neighboring companies during medium and good water periods. The present over-all efficiency for pumping (input to output) is 65% when the flow in the main river is below plant capacity at Stevenson, and 85.4% when the water in the main river would go over the Stevenson spillway without pumping. When the other projects below Rocky River are developed, the over-all efficiency will be 120%; that is, surplus water which requires the use of 1 kw-hr. to elevate it into the reservoir, when released will produce 1.20 kw-hr. in the Rocky River plant and down-stream plants when built. Hence, it is evident that whenever cheap energy or energy otherwise unusuable is available, it will pay to use it for pumping and thus decrease the average cost of energy in the system.

"In the future when additional peak capacity has been installed at Rocky River, such additional units may utilize water supplied by the pumps. Such

additional installation would perform the same peak load function as a Type 1 pumped-storage hydro-electric plant. The capacity required for the top 10% of the load is in use less than 100 hours per year. Without the pumping feature such additional contemplated cheap capacity at Rocky River would be inadvisable."

DISCUSSION

ROCKY RIVER POWER

The Rocky River plant, according to W. W. K. Freeman, Assoc. M. Am. Soc. C. E., was of particular interest because it was the first modern example in the United States of a type that has been under development in Europe for the past forty-six years. He described several European plants classified in accordance with the economic needs.

Handicaps to American Plants.—In commenting on Mr. Justin's paper, George A. Orrok, M. Am. Soc. C. E., brought out the point that "in most of the European installations the cost of the real estate has not been a serious matter." He stated that in the United States the real estate and water-right problems are difficult, and that in a dozen or more locations he had examined, where plants of this character might be installed, the real estate and water-right restrictions in most had been so severe that no profit could be shown.

Taking the pondage type of plant, that is, in which the released water is not used for regulation of other plants, it has generally been the case that a reasonable profit could be shown to the time that new steam machinery must be added to the system, the profits from the "water storage battery" being the coal saving during the time of operation plus the interest on the delayed construction. When the steam plant is at a distance from the water power, and high-tension transmission costs must be added, the profits fall off rapidly.

Mr. Orrok also called attention to the fact that bonds of power companies usually run forty or fifty years and the charges are not lowered by the scrapping of the older steam stations:

"In other words since interest has to be paid on the cost of the steam station that particular station will probably continue to be one of the cheapest methods of carrying peak loads."

He also stated that the duration curves shown by Mr. Justin appeared to be very flat, and that:

"Systems with the more usual load factor of 35 to 42% show no such high percentages in low-hour use. When the diagram is integrated for output and 93 to 95% of the output is generated on 50% of the apparatus, as in the case of systems having a large lighting and domestic service, the question of peaks becomes much more important and the question of firm power no longer appears. Where thunderstorm peaks occur, practically all power must be firm power or the system demand cannot be met."

Various Comments.—General remarks covering various features of the development were offered by E. J. Amberg, Esq., E. A. Van Deusen, M. Am. Soc. C. E., etc. E. J. Amberg, Electrical Engineer of the Connecticut Light

and Power Company, stated that while the plant had not been in full operation and exact comparative data were not available, the pumping was fully up to expectations and the leakage from the reservoir, which was one of the uncertainties, had been very small. He emphasized another point not previously brought out—that a hydro-electric plant held in reserve could be placed on service much more quickly than a steam plant.

It was pointed out by E. A. Van Deusen, M. Am. Soc. C. E., that a factor which added to the attractiveness of pumped-storage plants was the increase in the efficiencies of pumps and turbines from 75 to 80% for pumps ten years ago to 87% for pumps, and 90% for turbines, at the present time. He stressed the fact that conditions in Europe which favored the pumped-storage plant might not apply so well in this country, as load conditions might change so radically as to alter the situation on which the plant was based.

According to Carl P. Birkinbine, Esq., the economic factors should be studied carefully and every problem decided on its merits.

From another angle, L. F. Harza, M. Am. Soc. C. E., suggested that the term, "hydro-regenerating units," was better than "pumped-storage."

It would be difficult to combine the modern hydraulic electric unit and the pump remarked William M. White, M. Am. Soc. C. E., because a pump of high efficiency has a much higher speed for a given head than the ordinary turbine can supply. He also stated that efficiencies of 92% for turbines and 90% for pumps were obtainable.

WATERWAYS DIVISION

STATUS AND PROGRESS IN THE ART OF WATERWAYS ENGINEERING

REPORT OF THE EXECUTIVE COMMITTEE

This first report on the progress of the art by the Waterways Division is directed principally to a general résumé of the status of waterway development in the United States, and to the development of special features of design as exemplified in the New Welland Ship Canal.

Federal Work in the United States

General Investigations.—In 1927 Congress directed an extensive investigation of the principal navigable streams of the United States and of their tributaries, with the view to the formulation of general plans for their most effective improvement, for the purpose of navigation in combination with the most efficient development of potential water power, the control of floods, and the needs of irrigation. The Flood Control Act of 1928 extended the scope of the investigation to include tributaries of the Mississippi River System subject to destructive floods and authorized the expenditure of $5 000 000 for the purpose. These investigations are being actively prosecuted by the Corps of Engineers of the War Department, with the co-operation of the Geological Survey of the Interior Department.

Flood Control.—As a consequence of the disastrous flood of 1927, the Flood Control Act of 1928 has authorized the expenditure of $325 000 000 for the project for flood control in the alluvial valley of the Mississippi, in accordance with the engineering plan recommended by the Chief of Engineers in his report made at the opening of the 1927 session of Congress. The salient feature of this plan is the provision of floodways for the escape of the discharge in excess of the capacity of the levee system.

Navigation Works.—The project for providing navigation to a 9-ft. minimum channel depth on the Ohio River from Pittsburgh, Pa., to its confluence with the Mississippi, long in progress, is practically completed, and will be in full operation during the next low-water season. The adopted plan provided for fifty-four locks and movable dams. Modifications subsequently made, including the replacement of the four movable dams next below Pittsburgh with two fixed dams, have reduced the number to forty-nine. The lift at the dams in the system varies from about 6 to 12 ft., with the exception of Dam 41, at the Falls of the Ohio, which has a 37-ft. lift. At this site there is an hydro-electric plant with an installed capacity of 108 000 h.p., with an output of more than 400 000 000 kw-hr.

The Cape Cod Canal, originally constructed by private interests, has been acquired by the Federal Government. Work on a 27-ft. channel in the Hudson River, to Albany, N. Y., is well under way. A project for a 26-ft. channel in the San Joaquin River, to Stockton, Calif., to be prosecuted in co-operation with local interests, has been authorized by Congress. The improvement of the Missouri River for about 400 miles, from its mouth to Kansas City, is being actively prosecuted, and it is estimated that a depth of 6 ft. at low water will be secured by 1930.

Among important projects recommended to Congress for approval are the deepening of the interconnecting channels between the Great Lakes; the deepening of the channel over the bar outside San Francisco Harbor to 45 ft.; and a 35-ft. channel in the Columbia and Willamette Rivers, to Portland, Ore.

Illinois Waterway

This waterway will afford an 8-ft. depth, connecting Lake Michigan with the Illinois River. It is being constructed by the State of Illinois. Of the five locks on the project, each 600 ft. long and 110 ft. wide, with lifts ranging from 18 to 41 ft., three are completed, and work on the remaining two is in progress. Contracts covering other work have been let. The State has set April 1, 1931, as the date of completion and opening of the waterway.

St. Lawrence Waterway

Diplomatic correspondence has been carried on between the United States and Canada with reference to the construction of a deep-draft navigation channel between Lake Ontario and Montreal, Que., Canada, plans and estimates for which were presented by a Joint Board of Engineers in 1926-1927.

New Welland Ship Canal

Navigation from Lake Erie to Lake Ontario passes through the Welland Canal, constructed and operated by the Dominion of Canada. The present Welland Ship Canal under construction by Canada is 25 miles long and has seven locks, each with a lift of 46½ ft., and one guard-lock. The parts of the canal first excavated were given a depth of 25 ft.; the later contracts provide for a depth of 27 ft. The depth over the sills of the locks is 30 ft., to provide for future deepening of the canal without reconstruction of the locks. The importance of this project warrants a special description of its more novel features.

Locks.—The most important new feature in the design of the locks is the use of the high lift of 46½ ft. This is a considerable advance over lifts previously used in locks of this size. Coupled with the high lift a rapid rate of filling has been provided—as much as 8 ft. per min. at the beginning of the operation. While this rate is considerably greater than anything used heretofore in locks of this size, experience with the locks of the Panama Canal and of the Soulanges Canal indicate that no objectionable effect will follow.

As a result of this high lift special provision had to be made to facilitate the mooring of vessels in the locks at the lower pool level. This was done by providing at each side of the lock a mooring passage formed in the masonry

of the lock walls at a distance of 29 ft. below the coping level and extending from one end of the lock to the other, with openings out to the face of the lock wall at intervals of 60 ft. A mooring post is provided in each opening where a man can stand, catch, and make fast a mooring line from the ship. Access to the mooring passages, which will be submerged when the lock is filled, is by stairways from the coping level. Three stairways are provided to each mooring passage, one near each end and one at the center of the lock.

For the lock-valves the Taintor type was adopted after a thorough investigation of the relative advantages of the various types available. The Taintor valve was considered to be preferable to the Stoney valve on account of the frequency of operation required, which would produce rapid wearing of the Stoney rollers and treads, involving troublesome and expensive maintenance. Cylindrical valves of the necessary size would require larger valve chambers than could conveniently be provided. Furthermore, they could not be used as discharge valves without either placing them on a horizontal axis or using crooked and complicated water passages. The Taintor type was preferred to the butterfly type on account of the high heads to be carried, which would involve the use of rather unsatisfactory details in the design of valves of the necessary strength. Also, the high velocity of the water passing through the valves as a result of the high head would have produced objectionably unbalanced forces upon the valves when they were partly open.

Lock-Gates.—A radical departure from previous practice has been made in the design of the lock-gates by providing them with safety horns. These horns consist of steel castings placed at the miter end of each leaf and designed so that when one leaf of a gate is swung up stream from the mitered position through a distance of not more than 4 ft., the other leaf will still be supported against it by means of its safety castings. Without this device a mitering gate is easily unmitered under head and carried away by a slight blow from a ship on its down-stream side. With the safety castings, however, an upper gate carrying an assumed head of 15 ft. would not be unmitered if struck by a ship of 20 000 tons displacement, moving at a speed of 2 ft. per sec.; and a lower gate under its full head of 46½ ft. could withstand a blow from a ship of the same weight moving at a velocity of 5½ ft. per sec. The use of the safety horns increases the cost of the gates by approximately $360 000 for the entire canal.

Another unusual feature used in connection with the lock-gates is a curved concrete wall concentric with the pintle center placed on the floor of the lock under each gate leaf near its miter end. The top of the wall is reinforced with a grillage of three curved 12-in. I-beams, the top of the I-beams being placed about 4 in. from the bottom of the gate leaf. The purpose of this wall is to catch the toe of the leaf in the event of the anchorage being broken from any cause, and to prevent the quoin end of the leaf from swinging very far out of the hollow quoin. In this way the leaf would be held in the hollow quoin and be prevented from falling to the bottom of the lock unless it should happen to be near its open position.

The lock-gates are operated by wire ropes arranged in a new manner by taking advantage of this circular wall. The rope for opening the leaf is

anchored to the face of the lock-wall near the floor and is led along the convex vertical face of the circular wall to a sheave mounted on the bottom of the leaf. Similarly, the rope for closing the leaf is anchored to the sill of the gate and led along the convex vertical face of the circular wall to another sheave mounted on the bottom of the leaf. From these two sheaves the two operating ropes are led vertically up through the leaf and thence horizontally along its top to the quoin end, from which point they are led to the operating machine on the lock wall. This arrangement requires only two sheaves to be placed under water for each leaf and has a further advantage that the parts of the ropes which lead along a lock floor do not move, but simply lie against the circular wall as the sheaves roll along them.

Lock Fenders.—To protect the lock-gates from being struck by ships, fenders will be provided above and below the gates as required in different cases. Each lock fender will consist of a 3½-in. steel wire rope extending across the lock at a suitable distance from the gate which it protects and having a snubbing device at each end so that, if struck by a ship, the rope will pay out from either end under a heavy resistance and gradually stop the ship. To enable the fender to be opened in order to permit the passage of a ship, the rope will have a pin coupling at one side of the lock so that the part crossing the lock can be disconnected from the part engaging the snubbing device at that side of the lock. The part crossing the lock will be supported by a structural steel cantilever boom which can be swung up into a nearly vertical position like a bascule bridge, thus leaving the lock clear for the passage of a ship. The pin in the rope coupling will be withdrawn and inserted by mechanism actuated by a small electric motor.

Weirs.—The regulating weirs, in addition to providing for the flow over their crests of the feed-water required for regulating the levels of the various reaches between the locks, are furnished with large sluice-ways capable of discharging the great quantity of water necessary to prevent the canal banks from being overtopped in the event of the gates of a lock being accidentally carried away by a ship. As these sluice-ways will be operated very seldom, Stoney gates are provided to control them. Two sources of power are provided for operating the Stoney gates, namely, an hydraulic turbine and an electric motor; both sources of power are controlled automatically. When the water level above the weir rises to a predetermined stage it will flow over a small weir crest in the control chamber and fill a tank the weight of which, when full, will open a butterfly valve and allow water to flow through the turbine. If for any reason the turbine should fail to operate as intended, a further slight rise in the water level above the weir will start the electric motor by means of a float switch. This arrangement makes the opening of the sluice-ways in an emergency very reliable.

The completion of all concrete work and of practically all the earth and rock excavation is expected by April 1, 1930. The lock-gates are 40% erected, and it is expected that they will be fully completed by August 31, 1929. Work to date on the whole project indicates that unless something unforeseen develops that the whole of the Ship Canal will be in operation in 1930.

TERMINALS

The trend of development of water terminal facilities within recent years has been marked principally by a better recognition of the importance of such facilities at the river ports.

Little change has occurred in types of terminals at the seaports or those ports reached by sea-going vessels. They have been extended or enlarged as necessity required, and some new ones have been created. There has been, however, no radical departure from types of layout or construction which have been well known for many years. For the most part improvements have consisted in the use of more durable materials for construction, in the installation of additional mechanical devices for handling freight, and in developing better co-ordination between rail and water carriers.

The river ports have not been quite so nearly standardized, nor does it seem possible that they should be in the present state of the art. The varying fluctuation between high and low-water elevations, which may be only a few feet at one river port and as much as 70 or 75 ft. at another, and the wide range of topographical surroundings make it even more necessary with river ports than with seaports to deal with each port as an individual problem.

Respectfully submitted,

GEORGE B. PILLSBURY, *Chairman,*
W. H. MCALPINE, *Secretary,*
JOHN R. FREEMAN,
GEORGE F. NICHOLSON,
L. C. SABIN,
Executive Committee.

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W. H. McAlpine, *Secretary*,
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Executive Committee.

IRRIGATION DIVISION

STATUS AND PROGRESS IN THE ART OF IRRIGATION ENGINEERING *

By S. T. Harding,† M. Am. Soc. C. E.

To describe the status of irrigation requires a definition of the field of irrigation. It is assumed that this field includes such portions of the subject as fall within the scope of the Irrigation Division. While no attempt has been made by the Division to define or limit its field, its activities have included such subjects as financing, public policy, use of water, and legal administration, as well as the more technical engineering subjects. In consequence, a similarly broad definition of irrigation has been assumed.

Engineers engaged in irrigation are called on to determine agricultural features affecting the lands to be served, as well as to apply legal features regarding the acquirement of water rights and the procedure of organization, particularly of district or semi-municipal forms. In addition, engineers are called into more direct contact with financing in connection with feasibility than in many branches of engineering. The operation of the completed works is largely handled by engineers. All these fields as well as the design and construction are of interest and importance to irrigation engineers and are properly within the field of the Irrigation Division.

Agricultural Phases of Irrigation.—The recent tendency to emphasize the agricultural features which affect the utilization of the irrigation facilities to be provided for new lands has continued during the past year. The general lack of active demand for land resulting from the lower crop returns of recent years has continued, although in many areas this condition has improved somewhat during 1928.

The larger cost of available new projects is resulting in more careful land classification. While land of secondary quality may be acceptable in systems of low cost, only first-class land can support the higher cost of projects that require storage or other expensive works. The classification of irrigable lands includes the determination of both productivity and water requirements. The methods and standards of the former are similar to those used under other types of agriculture. For the latter, experience in practical irrigation is required, in addition to the knowledge of soils.

To bring lands into use more quickly, land preparation on a large scale in advance of settlement has been advocated and tried. Where the agricultural and engineering features of such land leveling are properly co-ordinated the results are better and cheaper. Large equipment for such work is economical

* This statement has been prepared by S. T. Harding, M. Am. Soc. C. E., at the request of the Executive Committee of the Irrigation Division. The time available has not permitted adequate assembling of widespread information. In consequence, the statements made rest mainly on the writer's own contacts, supplemented by such suggestions as he could secure.

† Prof. of Irrig., Univ. of California, Berkeley, Calif.

if the area to be prepared is great or if heavy grading is required. The most economical results are usually secured where the method of irrigation is fitted to the land rather than the land to some method of applying water. The individual land owner can frequently secure results more economically with his own equipment than may be obtained with larger organizations.

Studies of Water Resources.—The decreasing margin between total run-off and use and the realization that present development is approaching the limit which can be supported under unrelated projects is resulting in studies of the maximum use of available supplies. In California it has resulted in co-ordinated studies of the water resources of the entire State, financed by State appropriations and directed by the State Engineer. These studies, extending over several years, have cost about $500 000. Several reports on the results have been issued and much public interest has been aroused. Consideration is being given to the policy which should be followed in co-ordinating private development to produce the maximum benefits; and to the extent, if any, to which the State should participate either by controlling development or by actual financing thereof.

Similar studies are being undertaken or urged in other States. It is to be expected that the public interest in these subjects will increase and will result in added activity in such investigations of water resources; also, that there will be an increasing demand for greater co-ordination in the use of water. The extent to which this interest may be reflected in results will be affected by the conditions in the different States. Provisions for withdrawal of water from appropriation by legislative enactment or by authority granted to State officers have generally been made in those States the Constitutions of which do not prohibit the denial of the right of appropriation.

Such studies, including power, municipal, and mining purposes, are directed toward the most complete utilization of the available resources. It is generally recognized that it will not be practicable to bring about the most complete utilization of the larger streams in any single undertaking and there is an increasing public support for a policy of present partial development so as not to conflict later with such complete utilization. This tendency is found in the administration of water rights by the States to such extent as their present constitutional limitations permit as well as in the administration of public lands by the Federal Government.

At present, studies for water supplies and irrigation are more detailed than in the past. Consideration of return flow from present developments, recovery of percolation losses by pumping, and studies of the consumptive use by crops are illustrations. These tendencies are the natural result of the increasing difficulty of securing dependable additional water supplies with their higher resulting costs.

These detail studies have drawn attention to some features of water supply formerly largely neglected, such as the direct contribution of rainfall to the ground-water supply. While irrigation is practiced in areas of deficient rainfall so that penetration of rainfall is normally negligible, in some areas precipitation has resulted in direct penetration, as shown by detailed studies in Southern California during the past year.

Financing.—Difficulty in financing controls the extent of new irrigation development at present, except for projects receiving public aid. The difficulties of marketing securities based on irrigation alone is resulting in the combination, where available, of other uses with irrigation. Where the project involves storage, power may be obtainable. Several projects have thus been able to finance their development and an increased use of such combined sources of income is to be expected.

Irrigation has been affected by the same conditions in recent years that have affected agriculture generally. Projects started under the standards of costs and returns immediately following the close of the World War have had to adjust themselves to the new standards now applicable. In some cases the deflation of values has required adjustments in irrigation bonds similar to those necessary on land mortgages. Such adjustments are now under way in a number of the Western States.

Active discussion of public aid in financing irrigation development continues. There has been little change in regard to actual State aid during 1928. Adjustments of the losses where State aid has been extended have continued. Some additional adjustments by legislative authority will probably be sought at the 1929 legislative sessions.

These conditions have resulted in new development with private capital being limited to areas particularly favored as to soil, crop returns, or construction costs. During 1928 it has been confined mainly to additional areas under existing systems rather than to new projects. These limitations, particularly of costs, have not applied to the projects financed by Federal aid, which have continued their activities at rates similar to those of previous years.

Forms of Organization.—The most complex conditions surrounding new projects have required varied forms of organization. Such demand will probably continue. Several forms of district organization laws involving new methods of initiation, voting, and assessment are now in effect in California where the conditions are more variable and where many water developments have now reached a more complete stage. Differing tendencies continue, such as the change on Government projects from the contracts with individuals to the collective contracts with irrigation districts. In Oregon, the change from general liens to special or individual liens in irrigation districts is being actively advocated. Probably all States will need to amplify present laws covering organizations for irrigation to provide larger flexibility in their application.

Administration of Water Rights.—The administration of water rights by such legal authority as may control diversions from streams affects the operation of irrigation systems more closely than does the public supervision of other lines of engineering. A definite step toward co-ordinating such administration in the different States was taken in 1928 by organizing the Association of Western State Engineers. While the legal basis of water rights and the conditions to which they are applied differ, more uniform practices would be advantageous on many common problems. This Association represents an active effort to secure such advantages.

The various functions of the State and Federal Governments regarding lands and waters are much more important in the Western States having large areas of public lands and depending for their development on limited water supplies than in the Eastern States. The many problems of jurisdiction and control remaining to be adjusted, come within the field in which co-ordinated action by the States through their State engineers may be beneficial. The issues involved are more largely ones of public policy than of technical legal rights. On such issues the engineers are particularly well equipped to speak.

Dams.—The increased use of storage together with the large size of reservoir units required to make many storage sites feasible has maintained the recent interest in the design and construction of dams. The status of design was thoroughly covered in the Division program of the San Diego Meeting, in October, 1928.

The failure of the St. Francis Dam during 1928 has resulted in a very active interest in the public supervision of dams. Legislation to this end will probably be proposed in several States. Engineers are interested that proper provisions for public safety may be made without unnecessary restrictions which would handicap legitimate development. While dams affect several other branches of engineering, they are of particular interest to irrigation engineers, due to the extensive use of storage for irrigation.

Structures.—Many irrigation systems, particularly irrigation districts, follow a policy of gradual physical improvements, expending each year such amounts as may be obtained by direct assessment. Such improvements may consist of concrete lining of canals or the replacement of wooden structures. More attention is being given to the details of the smaller structures. Concrete turnouts, either of pipe or open-box type, are now more extensively used on the older systems than formerly. The designs of these standard structures are based on the local experiences with their earlier types which enables both permanence and better service to be secured.

Similar tendencies toward the use of permanent types of construction for diversion works are also apparent. Concrete construction is now more common for new works, although many wooden or temporary dams are still in use.

Measurement of Water.—While measurements of water diverted from streams are usual as a part of the administration of water rights, measurement of delivery to individual farms is not general, particularly on the older systems. Where costs are large, or the supply is limited, measurement is usual. The extension of such measurements has been retarded by the difficulty of securing dependable results without larger costs than many systems consider justified. In general, efforts to force such measurement on the basis of general advantages where not essential for local conditions have not produced satisfactory results.

During 1928, the use of the improved forms of Venturi flumes has increased for both main canals and individual deliveries. Experience with this device has shown its wide adaptability. It is now recommended by several State authorities.

Where the cost of a separate measuring structure for individual deliveries is not justified, many systems attempt to record delivery by operating the

turnout structure as an orifice. Care in maintaining standard conditions is required if dependable results are to be secured. Progress made in 1928 in developing such combined structures has included circular pipes rated for use as submerged orifices and combination turnout and Venturi meter types.

Ground-Water.—The use of ground-water developed by pumping from wells has been increasing rapidly since 1918. At first, it was confined mainly to California where ground-water basins of relatively large capacity occur and where climatic conditions enable crops of larger return to be produced, which conditions warrant larger costs for pumping. Pumping has been extended in some parts of California beyond the capacity of the supplies with consequent progressive ground-water lowering. Efforts to increase ground-water recharge are being made in many areas; in some other areas, the increase in pumping lift is resulting in reduction in draft.

The utilization of surface streams in other States has reached a point where development is turning toward ground-water. Several Western States are facing, or determining the basis of, titles to the use of ground-water for their areas. It is generally conceded that the old common law doctrine of entire ownership by the overlying land owner is not applicable to conditions of such extensive draft as required for irrigation. California met this by adopting through Court decision in 1903 the principle of correlative rights. Other States are now meeting this through provisions similar to those used for acquirements of rights to surface streams, or through such modifications of this procedure as may be considered applicable locally. Such problems promise to become of increasing importance in several States.

Drainage.—Drainage of irrigated lands is now generally regarded as part of irrigation. During 1928 there has been a continued activity in developing drainage by pumping from wells. Wherever the soil formations are sufficiently porous, pumping is proving well adapted to controlling the ground-water as well as for providing supplemental water supplies for irrigation. For lands having heavy texture to depths too great for effective results with either open or tile drains, drainage by pumping from wells is effective if such lands are underlaid by deeper strata sufficiently coarse to permit movement of ground-water to the wells. In soils of continuous heavy texture for considerable depth, all methods of drainage are difficult.

Summary.—As a whole, 1928 has not been an outstanding year from the point of view of irrigation. However, it has been a year of progress toward the establishment of better conditions on existing projects. If present developments can complete the utilization of their lands on a basis that is profitable to the land owners and, at the same time, meet their financial obligations, the future of irrigation can be faced with confidence. General conditions regarding such success are considered to have improved somewhat during 1928. There is no immediate prospect of any material stimulation in irrigation development, and it is probable that progress in 1929 will be similar to that in 1928. It is now considered by many of those concerned with irrigation that its permanent interests will be better served if undue stimulation, with its succeeding periods of depression, can be avoided and a steadier progress secured.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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PROGRESS REPORT OF THE SPECIAL COMMITTEE ON CONCRETE AND REINFORCED CONCRETE ARCHES *

To the Board of Direction,
American Society of Civil Engineers:

The Special Committee on Concrete and Reinforced Concrete Arches presents the following report of progress:

The work of the Committee during the past year has been confined almost entirely to the tests of arch ribs at the University of Illinois and to the analysis and preparation for publication of the data already obtained on arch ribs without superstructure.

Skew Arches

The tests on skew arch models† which were started at Princeton University, were temporarily discontinued because of lack of working space. It is hoped, however, that the finishing of the new Engineering Building will make possible their completion.

Points have been set in one of the piers of a 45°, skew-barrel, arch bridge in Columbus, Ohio, for observing the movements of the pier over a long period of time. The bridge consists of a series of arches from 75 to 95-ft. span and is built on a gravel foundation. Readings at weekly intervals, and at times of extremes of temperature, are being taken by the Engineering Experiment Station of the Ohio State University.

Laboratory Tests of Arches

The laboratory tests of arch ribs without decks, at the University of Illinois, have been completed,‡ and W. M. Wilson, M. Am. Soc. C. E., has analyzed the

* Presented at the Annual Meeting, January 16, 1929.

† Progress Report of Committee, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 167.

‡ *Loc. cit.*, p. 170.

data, the complete report being published as a *Bulletin* of the Engineering Experiment Station, University of Illinois. An abstract of this report is presented herewith.

LABORATORY TESTS OF ARCH RIBS WITHOUT DECKS

By W. M. Wilson, M. Am. Soc. C. E.

Description of Specimens and Apparatus.—The specimens were reinforced concrete arch ribs having a span of 17 ft. 6 in. and a rise of 4 ft. 0 in. These arches were tested in a 300 000-lb. Riehle machine. The methods of distributing the load and supporting the arch are apparent from Fig. 1. All supports were either knife-edges or steel balls; the abutments were supported in such a manner that all the horizontal thrust was taken by the tie-rods, *T R* 1, and measured by the springs, *S* 1; each vertical abutment reaction was divided into two parts that were weighted separately, hence their resultant was known in amount and position. The span of the arch was controlled by adjusting the nuts on the tie-rod, and the angular position of the abutments, by manipulating the jacks on the portable scales.

Some of the arches were subjected to symmetrical, others to unsymmetrical, loads. The apparatus shown produced a symmetrical load giving a thrust line lying within the kern of the rib over its entire length. For an unsymmetrical load, the arrangement was essentially the same, except that the moving of the bearing block for the head of the testing machine required the raising of the levers on one side and this, in turn, necessitated the transmission of the load from the I-beams by means of a heavy tie with suitable bearing. Such an arrangement gave a load 50% greater on the right than on the left half (Fig. 1), the purpose being to simulate the conditions of dead load over the entire span and live load over half the span, the rate of loading being twice as great for dead as for live load.

One feature of the loading apparatus is worthy of special note. The hangers not only had knife-edges top and bottom normal to the plane of the arch to insure proper division of the load, but they also had knife-edges top and bottom parallel to the plane of the arch to insure that the loading apparatus exerted no horizontal force on the structure. This feature is of special importance in the tests to determine the effect of the slenderness of a rib on its strength.

Object of Tests.—The objects of the tests were:

(*a*) To verify experimentally the elastic theory as applied to reinforced concrete arch ribs.

(*b*) To determine the relation between the strengths of concrete as developed in an arch rib and in control cylinders.

(*c*) To determine the effect of the slenderness ratio of an arch rib on the unit stress that it can develop.

Experimental Verification of the Elastic Theory.—The experimental verification of the elastic theory involves three parts:

1.—A comparison of theoretical and measured abutment reactions due to loads.

2.—A comparison of theoretical and measured stresses.

3.—A comparison of theoretical and measured reactions due to abutment movements.

The apparatus was planned to give the abutment reaction in magnitude, position, and direction. The diagrams of Fig. 2 give the relation between the theoretical and measured horizontal reaction for symmetrically and unsym-

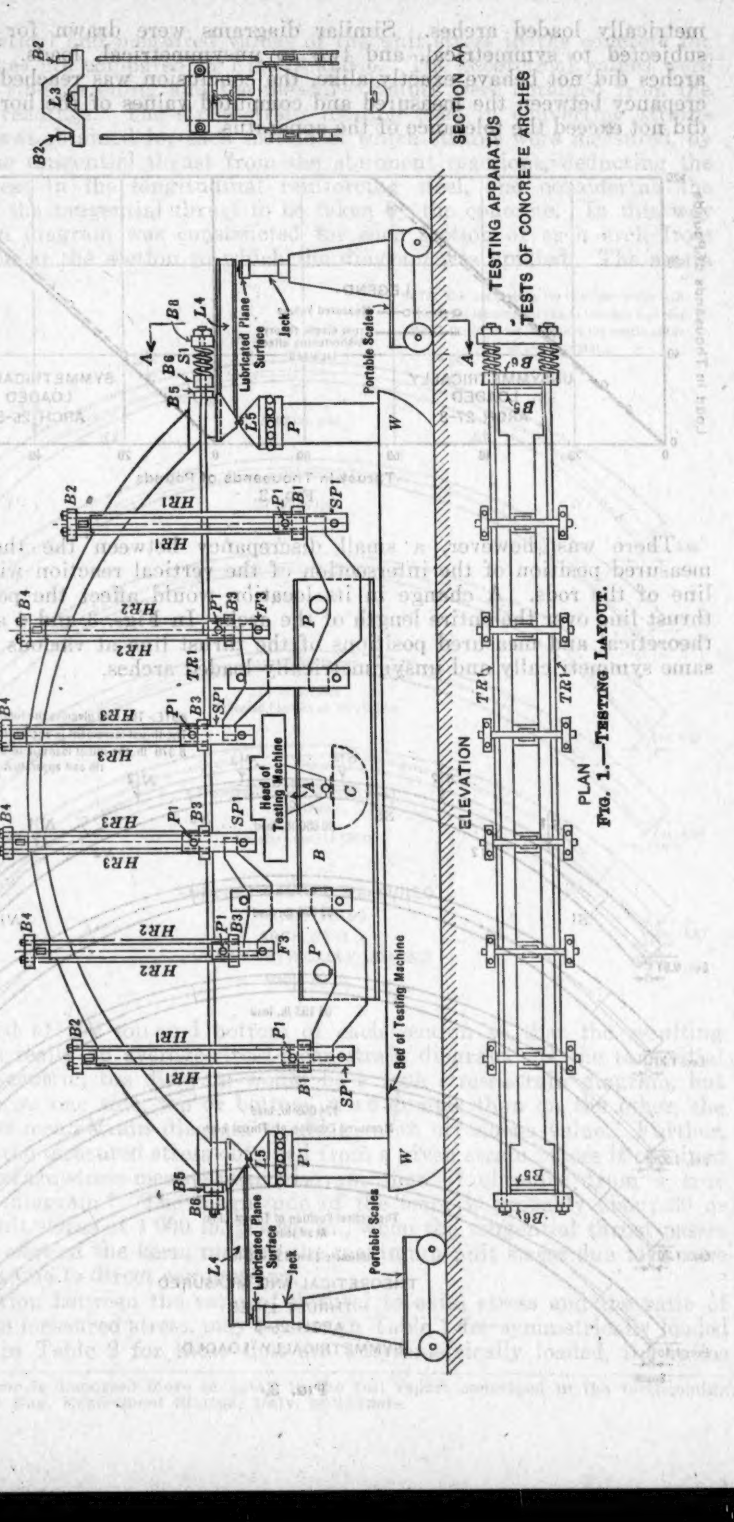


FIG. 1.—TESTING LAYOUT.

metrically loaded arches. Similar diagrams were drawn for seven arches subjected to symmetrical, and two to unsymmetrical, loads. Although all arches did not behave exactly alike, the conclusion was reached that the discrepancy between the measured and computed values of the horizontal thrust did not exceed the tolerance of the apparatus.

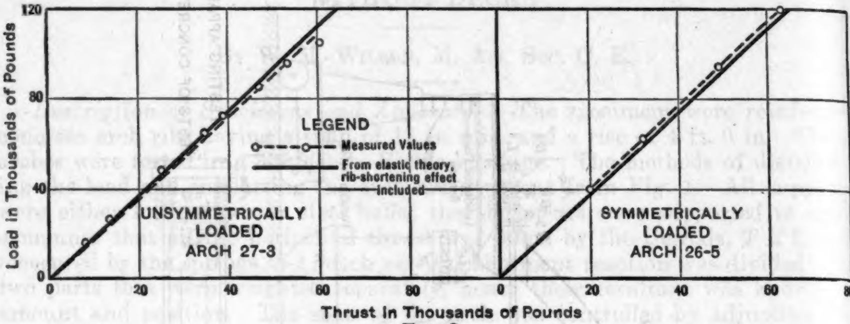


FIG. 2.

There was, however, a small discrepancy between the theoretical and measured position of the intersection of the vertical reaction with the center line of the rods. A change in its location would affect the position of the thrust line over the entire length of the arch. In Figs. 3 and 4 are shown the theoretical and measured positions of the thrust line at various loads for the same symmetrically and unsymmetrically loaded arches.

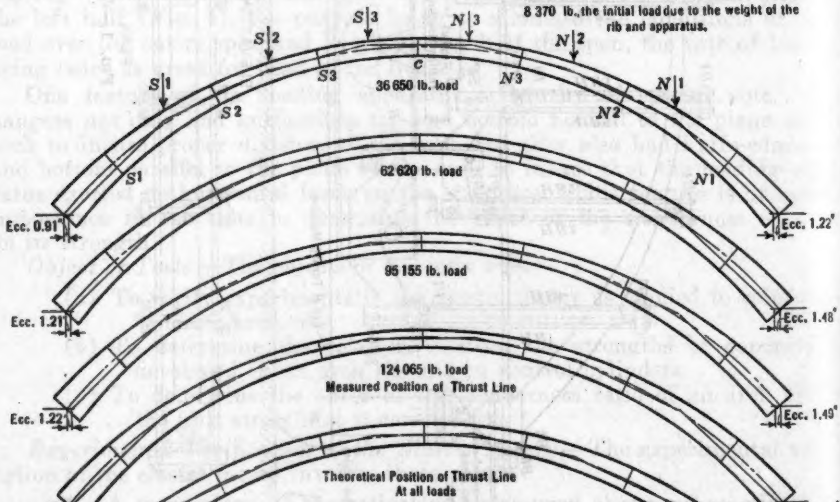
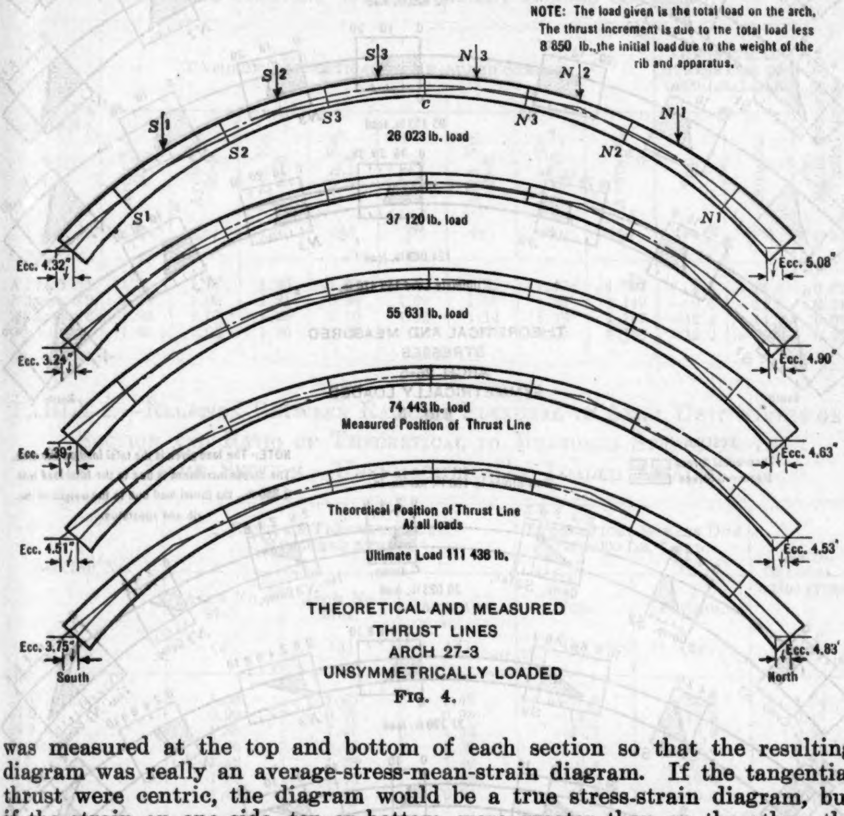


FIG. 3.

The theoretical and measured values of the unit stress in the concrete for the same arches are compared in Figs. 5 and 6. The theoretical values were corrected for rib-shortening and the measured values were obtained from the strain-gauge readings. The stress-strain relation used in converting strains into stresses was obtained for each section at which strains were measured, by computing the tangential thrust from the abutment reactions, deducting the measured stress in the longitudinal reinforcing steel, and considering the remainder of the tangential thrust to be taken by the concrete. In this way a stress-strain diagram was constructed for each section of each arch from readings taken at the section to which the diagram was applied. The strain



THEORETICAL AND MEASURED
THRUST LINES
ARCH 27-3
UNSYMMETRICALLY LOADED

FIG. 4.

was measured at the top and bottom of each section so that the resulting diagram was really an average-stress-mean-strain diagram. If the tangential thrust were centric, the diagram would be a true stress-strain diagram, but if the strain on one side, top or bottom, were greater than on the other, the average-stress-mean-strain diagram would give an erroneous value. Further, the value of the measured stress obtained from a given strain is less if obtained from an average-stress-mean-strain diagram than if obtained from a true stress-strain diagram.* The magnitude of the error is probably about 30 or 40%, at a unit stress of 1 000 lb. per sq. in., when the tangential thrust passes through the edge of the kern, making the maximum unit stress due to flexure equal to that due to direct compression.

The relation between the ratio of flexural to axial stress and the ratio of theoretical to measured stress, may be seen in Table 1 for symmetrically loaded arches and in Table 2 for those that are unsymmetrically loaded, the ratios

* This error is discussed more in detail in the full report contained in the forthcoming *Bulletin* of the Eng. Experiment Station, Univ. of Illinois.

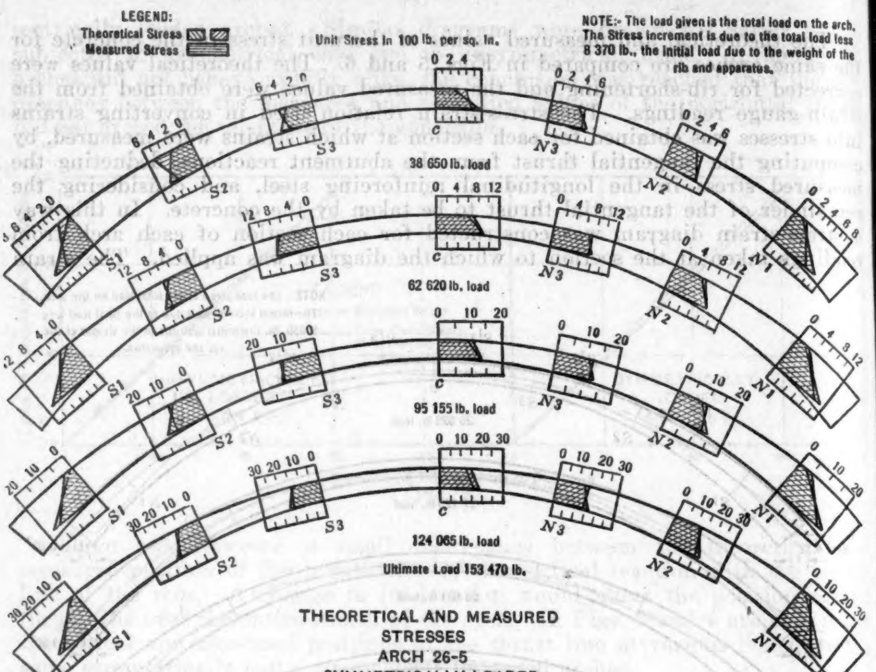

LEGEND:
Theoretical Stress
Measured Stress
Unit Stress in 100 lb. per sq. in.
NOTE:- The load given is the total load on the arch. The Stress increment is due to the total load less 8 370 lb., the initial load due to the weight of the rib and apparatus.
38 650 lb. load
62 620 lb. load
95 155 lb. load
124 065 lb. load
Ultimate Load 153 470 lb.
THEORETICAL AND MEASURED STRESSES
ARCH 26-5
SYMMETRICALLY LOADED
South
North

FIG. 5.

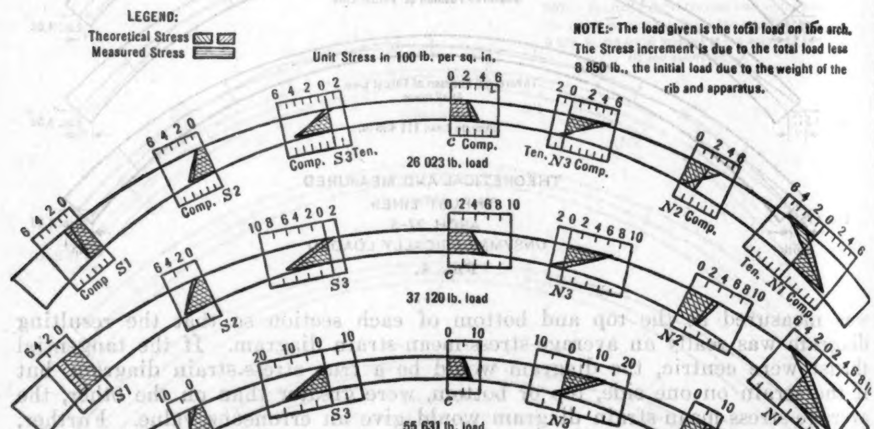

LEGEND:
Theoretical Stress
Measured Stress
Unit Stress in 100 lb. per sq. in.
NOTE:- The load given is the total load on the arch. The Stress increment is due to the total load less 8 850 lb., the initial load due to the weight of the rib and apparatus.
26 023 lb. load
37 120 lb. load
55 631 lb. load
74 443 lb. load
Ultimate Load 111 436 lb.
THEORETICAL AND MEASURED STRESSES
ARCH 27-3
UNSYMMETRICALLY LOADED
South
North

FIG. 6.

being given at five sections of each arch. The theoretical and measured values are in excellent agreement at those sections for which the moment is small; but the measured stress is less than the theoretical at those sections having a large flexural stress. If the error in the measured stress could be eliminated, the measured and theoretical values would agree more closely, for the sections subjected to a large flexural stress, than the diagrams indicate.

TABLE 1.—RELATION BETWEEN RATIO OF FLEXURAL TO AXIAL STRESS ON A SECTION AND RATIO OF THEORETICAL TO MEASURED STRESS ON SAME SECTION. SYMMETRICALLY LOADED ARCHES.

Section No.	Ratio of Theoretical to Measured Stress.								Theoretical Stress Due to 1 000-Lb. Load.		Ratio, flexural to axial stress.
	Arch No. 26-1.	Arch No. 26-2.	Arch No. 26-3.	Arch No. 26-4.	Arch No. 26-5.	Arch No. 26-6.	Arch No. 26-10.	Average.	Top.	Bottom.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
*N*1 and *S*1.	1.36	1.37	1.35	1.35	1.14	1.32	1.47	1.337	− 1.5	−21.5	0.87
*N*2 and *S*2.	1.34	1.09	1.21	0.98	1.09	1.11	1.22	1.149	− 9.6	−14.8	0.21
*N*3 and *S*3.	1.22	1.07	1.20	1.10	1.12	1.14	1.18	1.147	−12.4	−14.2	0.07
Center......	1.00	1.06	1.60	1.10	1.11	1.27	1.04	1.168	−12.0	−15.6	0.15

TABLE 2.—RELATION BETWEEN RATIO OF FLEXURAL TO AXIAL UNIT STRESS ON A SECTION AND RATIO OF THEORETICAL TO MEASURED STRESS ON THE SAME SECTION. UNSYMMETRICALLY LOADED ARCHES.

Section No.	Ratio of Theoretical to Measured Stress.			Theoretical Stress Due to 1 000-Lb. Load.		Ratio, flexural to axial stress.
	Arch No. 27-3.	Arch No. 27-4.	Average.	Top.	Bottom.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
*N*1..................	1.53	1.43	1.48	+ 7.9	−32.2	1.64
*N*2..................	0.99	0.97	0.98	−14.5	−10.3	0.17
*N*3..................	1.38	1.47	1.43	−25.7	− 0.9	0.93
Center	1.06	0.94	1.00	−12.2	−15.4	0.12
*S*3..................	1.42	1.38	1.40	+ 0.4	−27.0	1.03
*S*2..................	1.30	1.13	1.22	− 5.0	−19.0	0.58
*S*1..................	1.02	0.94	0.98	−11.0	−10.6	0.02

Diagrams similar to those in Figs. 5 and 6 were drawn for three symmetrically loaded and two unsymmetrically loaded arches. They are not exactly alike, yet all show about the same discrepancy between theoretical and measured values.

The effect of abutment movements on reactions was determined by measuring the changes in the reactions due to moving the abutments definite predetermined amounts, the total load on the arch remaining constant. There were four steps in this operation:

1.—The arch was loaded, the abutments being restrained, and all reactions, horizontal thrust, vertical reactions, and moment at the springing line, were recorded.

2.—The abutments were spread, the abutment rotation and the change in load being kept as small as possible.

3.—The north abutment was rotated, the span, load, and angular position of the south abutment being kept as near constant as possible.

4.—The south abutment was rotated, the span, load, and angular position of the north abutment being kept as near constant as possible.

The observations made after each of these movements, gave data from which the following quantities could be computed:

(*a*) Thrust and moment at springing line due to unit change in span.

(*b*) Thrust and moment at nearer springing line, and moment at farther springing line due to unit rotation of north abutment.

(*c*) Thrust and moment at nearer springing line, and moment at farther springing line due to unit rotation of south abutment.

The theoretical value of these quantities depends on the value of E for concrete. The tests have been interpreted by computing a composite E (Tables 3 and 4), which, if it were constant over the whole arch, would make equal the theoretical and measured reactions resulting from abutment displacements.

TABLE 3.—VALUES OF COMPOSITE E OBTAINED FROM VARIOUS RELATIONS.

Relation from which E was determined. (1)	Maximum E. (2)	Minimum E. (3)	Average of five tests, E. (4)
(1) Moment at springing due to change in span	3 124 000	2 153 000	2 546 000
(2) Horizontal thrust due to change in span	3 575 000	2 827 000	3 169 000
(3) Horizontal thrust due to rotation of an abutment	2 809 000	1 863 000	2 564 000
(4) Moment at a springing due to the rotation of the adjacent abutment	2 772 000	2 125 000	2 390 000
(5) Moment at a springing due to the rotation of the opposite abutment	3 599 000	2 937 000	3 346 000

TABLE 4.—AVERAGE VALUES OF COMPOSITE E FOR VARIOUS ARCHES.

Arch No.	Load, in pounds.	Approximate average unit stress, in pounds per square inch.	Composite E, average of values obtained from five relations.	Strength of concrete as determined from tests of cylinders.
26-6	18 920	240	2 758 000*	2 918
26-6	22 440	280	2 605 000*	2 918
26-7	21 045	267	2 958 000	3 112
26-7	41 085	516	3 080 000	3 112
26-8	39 660	500	2 614 000	2 452

* Compare with value of 2 540 000 from Table 7.

The modulus of elasticity of the concrete in Arches 26-7 and 26-8 (Table 4) was not determined from strain-gauge readings, but the average value for all arches as given in Table 7 is 2 500 000 lb. per sq. in. When differences in stress are considered, the composite E determined from measured abutment reactions and moments, was found to agree very closely with the average E determined from strain-gauge readings at the top and bottom of seven sections of the arch.

The computations for determining the effect of abutment movements on reactions were based on the assumptions that E for the concrete has the same value at all sections and at all loads and, in determining the moment of

inertia of a section, that the concrete resists tension. The computations for the horizontal thrust and for the theoretical stress due to load, were based on the same assumptions. The close agreement between theoretical and measured values, when the former were computed on this basis, would seem to justify the following practice:

> In analyzing an arch to determine the thrust and moment at the various sections, E may be assumed to have the same value at all sections and all loads, and the moment of inertia of a section may be based on the assumption that the concrete takes tension. It does not follow, however, that the moment of inertia used in computing the unit stress on a section due to a specified moment should be based on the same assumptions.

Relation Between Strengths in an Arch Rib and in Control Cylinders.—The relation between the strength of concrete as developed in an arch and that of the same concrete as developed in a 6 by 12-in. control cylinder is of vital importance because of its influence on the unit stress to be used in the design of an arch. An accurate comparison of these strengths, however, was difficult because of the many factors affecting them. The large variations in the modulus of elasticity at the different sections of the same arch indicate that the concrete was not uniform in quality even if all parts of the arch were poured from the same mix and on the same day. For this reason the modulus of elasticity, as well as the ultimate unit stress, was used in studying the properties developed by the material in the arch and in the cylinder.

The theoretical unit stress corresponding to the ultimate load, at each of seven sections, is given in Tables 5 and 6. Table 7 shows the modulus of elasticity of the concrete at the same sections. Not a single arch broke at a section of maximum stress, but seven of the nine arches failed at a section for which the modulus of elasticity was a minimum. The exceptions were Arches Nos. 26-4 and 27-4. Arch No. 26-4 broke at the center where E was 1 910 000 instead of at Section $N3$ where it was 0.5% less and the theoretical stress was 10% less than at the center. Arch No. 27-4 broke at Section $S3$ where E was 2 150 000, instead of at the center where it was 15% less, and the theoretical stress was 45% less than at Section $S3$. That is, for the two arches that did not fail at the section of minimum E, the minimum E occurred at a section where the unit stress due to a given load was correspondingly lower than the modulus of elasticity.

The great variation in the modulus of elasticity of the concrete in an arch and the fact that the arches in general broke at sections of small E rather than at those of large unit stress, emphasizes the necessity of placing concrete in the same manner over an entire specimen. The experimental arches were poured in a vertical position. The sides and bottom of the forms were in place when pouring began. Pouring started at the two ends simultaneously, and the top of the form was added as pouring progressed. It was difficult to place the concrete near the ends, but more easy toward the center. The concrete hardest to place received the most attention, and the consolidation was greater at the ends than at any other point, the degree of consolidation decreasing toward the crown. This is apparent from the values of E given in Table 7.

The unit stress for the arch (Column (2), Table 8) is the theoretical stress at the section where failure occurred, corresponding to the ultimate load; the unit stress for the cylinder (Column (3)) is the average value obtained from tests of three cylinders for each arch, poured from batches that went into the structure. The modulii of elasticity of the concrete in the arch and in the cylinders are also given in Table 8. Only arches 6 in. or more in width were included in this comparison.

The strengths developed by the concrete in the arch and in the cylinder were almost the same for Arches Nos. 26-1, 26-2, 26-5, and 27-4. For Arch

TABLE 5.—ULTIMATE LOAD AND MAXIMUM THEORETICAL UNIT STRESS FOR SYMMETRICALLY LOADED ARCHES.

Arch No. (1)	Width of rib, in inches. (2)	Point of failure. (3)	Ultimate load, in pounds. (4)	Section No. (5)	Unit stress at ultimate load by elastic theory, in pounds per square inch. (6)
26-1	8	Center	193 820	N 1 and S 1	3 396
				N 2 and S 2	2 351
				N 3 and S 3	2 254
				Center	2 479*
26-2	6⅛	Load Point S 2	153 110	N 1 and S 1	3 494
				N 2 and S 2	2 428*
				N 3 and S 3	2 318
				Center	2 557
26-3	4½	Center	84 800	N 1 and S 1	2 636
				N 2 and S 2	1 834
				N 3 and S 3	1 744
				Center	1 942*
26-4	3⅛	Center	65 700	N 1 and S 1	2 936
				N 2 and S 2	2 032
				N 3 and S 3	1 952
				Center	2 169*
26-5	6⅛	Center	153 470	N 1 and S 1	3 300
				N 2 and S 2	2 271
				N 3 and S 3	2 179
				Center	2 394*
26-6	6⅛	Center	159 450	N 1 and S 1	3 428
				N 2 and S 2	2 360
				N 3 and S 3	2 264
				Center	2 487*
26-10	6⅛	Load Point S 4	147 860	N 1 and S 1	3 179
				N 2 and S 2	2 188
				N 3 and S 3	2 100
				Center	2 307*

* Sections where failure occurred.

TABLE 6.—ULTIMATE LOAD AND MAXIMUM THEORETICAL UNIT STRESS FOR UNSYMMETRICALLY LOADED ARCHES.

Arch No. (1)	Width of rib, in inches. (2)	Point of failure. (3)	Ultimate load, in pounds. (4)	Section No. (5)	Unit stress at ultimate load by elastic theory, in pounds per square inch. (6)
27-3	6⅛	Load Point N 4	111 436	N 1	3 600
				N 2	1 616
				N 3	2 864
				Center	1 716*
				S 3	3 009
				S 2	2 117
				S 1	1 226
27-4	6⅛	Section S 3	135 917	N 1	4 390
				N 2	1 971
				N 3	3 493
				Center	2 093
				S 3	3 670*
				S 2	2 582
				S 1	1 495

* Sections where failure occurred.

TABLE 7.—Secant Modulus of Concrete in Arch at Stress of 1 000 Pounds per Square Inch, in 1 000 000 Pounds per Square Inch.

Arch No.	Section.							
	$N1$.	$N2$.	$N3$.	Center.	$S3$.	$S2$.	$S1$.	Average.
Symmetrically Loaded Arches								
26-1	3.12	2.73	2.41	2.25*	2.48	2.48	4.31	2.81
26-2	3.34	3.00	2.53	2.63	3.26	2.50*	3.25	2.93
26-3	3.45	2.28	2.21	1.18*	1.50	1.54	3.25	2.20
26-4	2.33	2.18	1.90	1.91*	2.16	2.10	2.55	2.16
26-5	3.92	2.56	2.46	2.06*	2.30	2.34	2.79	2.63
26-6	3.28	2.80	2.12	1.74*	2.50	2.36	2.99	2.54
26-10	2.62	1.70	1.88	1.48*	2.11	2.21	2.75	2.11
Average..........	....	....	....	....	....	....	....	2.48
Unsymmetrically Loaded Arches								
27-3	4.32	2.06	2.08	1.38*	1.62	1.93	1.74	2.16
27-4	5.41	2.42	2.25	1.82	2.15*	2.78	2.63	2.78
Average..........	....	....	....	....	....	....	....	2.47

* Sections at which failure occurred.

TABLE 8.—Comparison of Unit Stresses Developed in Arches and in Control Cylinders.

Arch No.	Unit Stress, in Pounds per Square Inch.		Secant Modulus at 1 000 Pounds per Square Inch.‡			Section having maximum E.†
	Arch.	Cylinder.	Arch at section of failure.	Cylinder.	At section where it is a maximum.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
26-1	2 479	2 563	2.25	3.51	4.31	$S1$
26-2	2 428	2 590	2.50	4.08	3.84	$N1$
26-5	2 394	2 470	2.06	2.76	3.92	$N1$
26-6	2 487	2 990	1.74	2.94	3.28	$N1$
26-10	2 307	1 396*	1.48	1.41	2.75	$S1$
27-3	1 716	3 380	1.38	2.95	4.32	$S1$
27-4	3 670	3 575	2.15	* 4.00	5.41	$S1$

* This is the value reported, but it is apparently in error.

† The maximum unit stress occurs at Sections $N1$ and $S1$ for all arches of the 26-series and at Sections $N1$ for those of the 27-series.

‡ Secant modulus is in 1 000 000 lb. per sq. in.

No. 26-10, Table 8 shows a greater strength for the concrete in the arch than in the cylinder, but the results of the cylinder tests appear to be in error. For Arches Nos. 26-6 and 27-3 the unit stress developed in the arch was much less than that developed in the cylinder. The failure of Arch No. 27-3 would seem to have been due to the poor concrete at the center ($E = 1\,380\,000$) since the arch developed a stress (theoretical) of 3 000 lb. per sq. in. at Section $S3$. Arch No. 26-6 failed at the center where the modulus of elasticity

was 1 740 000, but developed a stress of 3 428 lb. per sq. in. at Sections $N1$ and $S1$ where the modulus of elasticity was 3 280 000 and 2 990 000 lb. per sq. in., respectively. If the modulus of elasticity is accepted as an indication of the strength of concrete, the failure of all low-strength arches can be attributed to weak concrete at the point of failure. Corresponding unit stresses (Columns (2) and (3), Table 8) differ greatly for some arches; but when the unit stresses are studied in conjunction with the modulii of elasticity, the differences in strength would seem to be affected more by differences in the quality of the concrete than by differences in the shape of the structure, arch, or cylinder.

The results presented in Tables 5, 6, 7, and 8 seem to justify the belief that concrete will develop approximately the same strength in an arch and in a control cylinder providing it is consolidated to the same degree in both structures.

By far the most important conclusion to be drawn from these data is that:

> The strength of an arch is determined by the strength of its weakest section, and failure to consolidate the concrete at a single section may be as serious as a partial omission of cement, as the use of poor aggregate, or as the use of too much water.

The arches in these tests were poured more carefully than many concrete structures, yet the variations in the modulus of elasticity and, presumably, in the strength, were very great.

Effect of the Slenderness of an Arch Rib Upon its Strength.—Arches Nos. 26-1, 26-2, 26-3, and 26-4 were used in the study to determine the effect of the slenderness of a rib on its strength. They were identical in profile, all had the same percentage of reinforcement, and all were subjected to the same kind of loading; but they differed in width (8, 6⅛, 4½, and 3⅛ in., respectively). As already noted, the loading hangers offered no resistance to horizontal motion of the arch.

The relation between the load per inch width of arch and the lateral deflection at the crown is given in Fig. 7. The lateral deflection increased slightly with the load up to the ultimate for Arches Nos. 26-1 and 26-2; there is a considerable increase in the lateral deflection at high loads for Arch No. 26-3, although the failure did not partake of the character of buckling; but the lateral deflection was very large for Arch No. 26-4 and the failure was a typical buckling failure. Data pertaining to these arches are given in Table 9.

TABLE 9.—EFFECT OF SLENDERNESS RATIO ON STRENGTH OF AN ARCH RIB.

Arch No.	Width, in inches.	Slenderness ratio.	Ultimate load per inch width, in pounds.	Secant modulus at 1 000 lb. per sq. in. stress at section of failure.
26-1	8	30	24 280	2 250 000
26-2	6⅛	39	25 000	2 500 000
26-3	4½	53	18 840	1 118 000
26-4	3⅛	76	21 000*	1 910 000

* Buckling began at 18 700 lb. per in. width of rib and, if this load had been retained, failure would probably have eventually resulted without further increase in the load.

> The results of these tests would seem to justify the conclusion that ribs having a ratio of unsupported length to width of 30 are not weakened by a tendency to buckle as long as they are not subjected to the action of lateral forces. Practical consideration should prevent the use of ribs having a greater slenderness ratio.

Conclusions.—The results of the tests on reinforced concrete arch ribs seem to justify the following conclusions:

1.—The measured and theoretical values of the reactions due to load, both moment and thrust, are in close agreement.

2.—The measured and theoretical values of the reactions due to abutment displacement are in close agreement for these arches if the theoretical value is based on a modulus of elasticity equal to that of the concrete in the arch.

3.—The measured and theoretical values of the unit stress due to load agree closely.

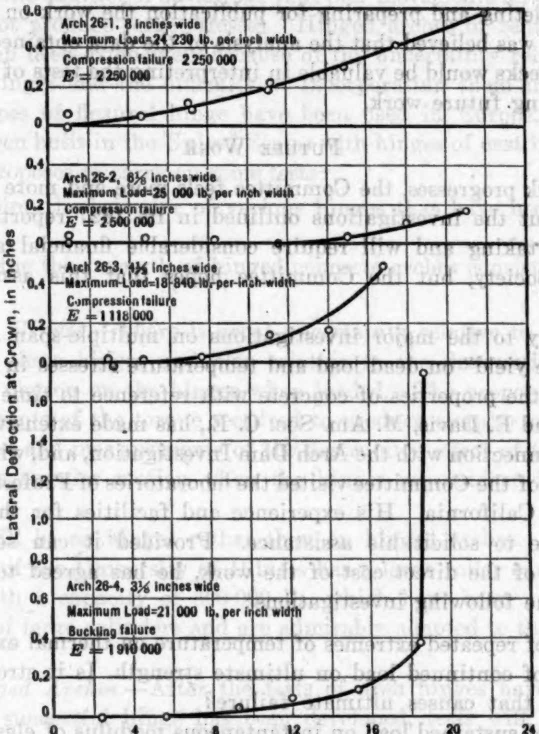


FIG. 7.

4.—The unit stress developed by the concrete in the arches was approximately equal to that developed by the same concrete in 6 by 12-in. control cylinders.

5.—A considerable local variation in I or E does not materially affect the behavior of an arch rib.*

6.—In analyzing an arch to determine the thrust and moment at various sections, the modulus of elasticity of the concrete may be assumed to have the same value at all loads and at all sections; and the moment of inertia of a section may be based on the assumption that the concrete resists tension. The errors involved will not materially affect the value of the moment and thrust thus determined. It does not necessarily follow, however, that the stress at a given section due to a specified moment should be based on the same assumptions.

* This statement is based on an analytical study by Hardy Cross, M. Am. Soc. C. E., in the forthcoming *Bulletin* of the Eng. Experiment Station, Univ. of Illinois.

7.—If an arch rib is not subjected to lateral forces, its strength does not seem to be affected by its slenderness as long as the ratio of the unsupported length of the arch axis to the width of the rib is less than 30.

Laboratory Tests of Arches with Decks

The tests at the University of Illinois on arch ribs with decks, as outlined in the 1927 report of the Committee,* have been delayed somewhat by the work of completing and preparing for publication the work on arch ribs without decks. It was believed that the analysis of the data obtained regarding the ribs without decks would be valuable in interpreting the tests of ribs with decks and in planning future work.

Future Work

As the work progresses, the Committee feels more and more the importance of carrying out the investigations outlined in its 1927 report.* This will be a large undertaking and will require considerable financial assistance from outside the Society, but the Committee hopes that this assistance will be forthcoming.

Preliminary to the major investigations on multiple-span arches and the effect of "time yield" on dead load and temperature stresses in arches, further knowledge of the properties of concrete with reference to time yield is necessary. Raymond E. Davis, M. Am. Soc. C. E., has made extensive researches in this field in connection with the Arch Dam Investigation, and, with this in mind, the Chairman of the Committee visited the laboratories of Professor Davis at the University of California. His experience and facilities for this research led the Committee to solicit his assistance. Provided it can secure funds to defray a part of the direct cost of the work, he has agreed to undertake all or a part of the following investigations:

1.—Effect of repeated extremes of temperature on thermal expansion.

2.—Effect of continued load on ultimate strength. Is it stress or deformation that causes ultimate failure?

3.—Effect of sustained load on instantaneous modulus of elasticity.

4.—Flow due to alternating stresses, say, 0 to 600 to 0 to 600 lb. per sq. in., etc.

5.—Flow due to alternating stresses, say, 300 to 600 to 300 to 600 lb. per sq. in., etc.

6.—Flow of dry concretes, humidity 50%, or less.

7.—Release of stress due to flow, maintaining deformation constant.

8.—Effect of reinforcement on flow. Bond flow.

9.—Effect of age at loading.

Charles Derleth, Jr., M. Am. Soc. C. E., Director of the Testing Laboratories of the University of California, has tendered the use of the laboratories and equipment, and has endorsed the proposal of Professor Davis. These investigations of time yield will be of equal importance to all branches of

* *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 167.

reinforced concrete design. This work and the continuation of the studies on single span arches with decks at the University of Illinois, the Committee is asking the Society, with the assistance of Engineering Foundation, to finance during 1929.

Hinged Arches

Theoretically, a hinged arch has most of the advantages of the arch without hinges, and it has the additional advantage that pier movements and changes in temperature do not produce large stresses. Hinged arches of reinforced concrete have not been used extensively because of the uncertainty relative to the behavior of the hinges and the difficulty of incorporating them in the structure. Several types of flexural hinge have been used in Europe, and a few structures have been built in the United States with hinges of cast iron or steel. The Committee proposes to carry on some tests:

1.—To determine the behavior of various hinges that have been and may be devised; and

2.—To determine the strength of hinged concrete arches consisting of a rib and deck.

Tests of Arch Hinges.—There is great uncertainty relative to the friction required to turn these hinges and also relative to the destructive effect of repeated angular motion on the hinges when loaded. The proposed tests will include measurements of the torque required to produce angular motion when the hinge is loaded and measurements of the destructive effect of thousands of repetitions of the angular motion when the hinges are subjected to various loads.

These tests will be on hinges rather than on hinged arches. The Structural Laboratory of the University of Illinois has a large roller carriage and a roller bearing with a capacity of 600 000 lb., which have been used in making rolling tests of large cylinders and are admirably adapted to these proposed tests.

Tests of Hinged Arches.—After the tests of arch hinges have been completed, and if a successful hinge has been developed, tests will be made on hinged arches in which the rib and deck are monolithic. By planning the rib, columns, and deck as a single structure and by properly locating the expansion joints in the deck, the arch may be designed as a Vierendeel truss having a curved bottom chord, and full advantage can be taken of the moment-resisting ability of this type of structure. Various designs will be tested in order that the measured and theoretical values of the strength of this type of structure may be compared.

Abutment movements produce practically no stress in three-hinged arches. If, therefore, the theoretical strength is verified by tests and if an economical and thoroughly reliable hinge can be developed, this type of structure, at present practically unused, should become quite common for highway bridges of medium span.

The fact that the piers of reinforced concrete arch bridges do move under stresses produced by temperature changes, unbalanced thrusts, and other

causes, makes the consideration of the stresses due to these movements important. True, if the piers are founded on rock, the movements are small, but the measurements taken on the bridge at Danville, Ill.,* prove that even then the stresses produced are of some magnitude. If, however, the piers are founded in gravel or some other yielding material, as is often the case, some practical method of relieving the stresses due to pier movements would seem to warrant the proposed study of arch hinges.

Respectfully submitted,

CLYDE T. MORRIS, *Chairman*,
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GEORGE E. BEGGS, *Secretary*.

December 5, 1928.

* Progress Report of Committee, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 168; *Bulletin No. 174*, Eng. Experiment Station, Univ. of Illinois.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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FINAL REPORT OF THE SPECIAL COMMITTEE ON IMPACT IN HIGHWAY BRIDGES *

To The American Society of Civil Engineers:

The Committee was appointed in 1922, and was asked to search for available data on the subject of impact in highway bridges, to encourage experimental work in a more systematic manner than had yet been attempted, and to report to the Society concerning the available information and its application to the design of highway bridges.

Subsequently, progress reports have been presented at each Annual Meeting and published in *Proceedings*.† Two of these reports‡ contain pertinent data of value.

This report undertakes to present such conclusions as the Committee has been able to make from the available data; also, in the form of appendices, abstracts of such data as will assist those who are especially interested to make independent applications of the information and to direct future work along the most profitable lines.

The only data published prior to 1922, so far as the Committee could ascertain, consisted of a number of theoretical discussions and the results of experimental work by F. O. Dufour,§ M. Am. Soc. C. E., made in Illinois from 1910 to 1912. By reason of the revolutionary changes in highway transportation and in the design of highway bridges that have taken place since these experiments were made, the data obtained at that time are of but little value in the present study, and have been utilized only in the construction of Fig. 1. The only other useful information which the Committee has been able to find is from two sources:

* Presented at the Annual Meeting, January 16, 1929.

† *Proceedings*, Am. Soc. C. E., March, 1923, Papers and Discussions, p. 457; March, 1924, Society Affairs, p. 260; March, 1925, Society Affairs, p. 126; March, 1926, Papers and Discussions, p. 442; March, 1927, Society Affairs, p. 120; and March, 1928, Society Affairs, p. 181.

‡ *Loc. cit.*, March, 1923, Papers and Discussions, p. 457, and March, 1926, Papers and Discussions, p. 442.

§ *Loc. cit.*, October, 1926, Papers and Discussions, pp. 1737–1742; *Journal*, Western Soc. of Engrs., Vol. 18, 1913.

(1) Studies on highway bridge impact at the Iowa State College, Ames, Iowa. These studies were begun in 1922 as a co-operative project between the U. S. Bureau of Public Roads, the Iowa Highway Commission, and the Engineering Experiment Station of Iowa State College. In 1924, very considerable assistance was rendered directly by the Society, through this Committee, by providing for its use a McCollum-Peters electric telemeter,* or strain-gauge, for determining and recording various stress measurements. Reports from this project have been published.†

(2) Experiments of the U. S. Bureau of Public Roads on impact on pavements. These results have also been published.‡

The report is primarily an effort to analyze and correlate the data obtained from these projects and to draw such conclusions as seem to be warranted at this time.

Nature of the Problem

Obviously, the most direct method of determining dynamic stress and, therefore, impact in a structure is by direct measurement of such stress under moving loads subject to the various conditions met in practice. To a certain extent this is practicable, and much information of this nature was obtained in the tests at Ames, Iowa. This method, however, has its limitations. Not only are the variations of loading, truck design, and tire conditions very great and not easily defined and related, but the structure itself introduces many complications. Smoothness of floor, nature of possible obstructions to be anticipated, the design of the structure itself, and the length of roadway to be loaded for maximum effect, all enter into the problem and seriously complicate it. To cover all these variables by tests in the field is practically impossible. In particular is it difficult to determine the significant impact for those members of a structure that require a considerable load. As a consequence, the tests at Ames were directed primarily to floor-beams and stringers. Impact values were determined in a few truss members of a number of bridges, but the results are of less importance, as in most cases the simultaneous unit stresses were very low.

The greatest impact of a wheel passing over an obstacle may be from shock as the wheel strikes the obstacle, or from drop as it again strikes the floor. According to the available data, drop impact is generally the greatest for heavily loaded trucks. Shock impact was frequently greatest for unloaded trucks, but the resultant stresses were so low as to suggest that shock impact is of theoretical, rather than practical, interest.

As compared to railway bridge impact, the problem is a very different one, especially as regards truss members. In railway bridges, the impact effect arises largely from the rotation of unbalanced drivers; and synchronism of driver rotation and bridge vibration produces dynamic stresses which are subject to certain fairly well-defined laws. Such synchronism of impact can

* *Bulletin No. 247*, U. S. Bureau of Standards.

† *Bulletins 63* and *75*, Eng. Experiment Station, Iowa State Coll.; *Public Roads*, September, 1924.

‡ *Public Roads*, March and December, 1921, June, 1926, and August, 1928.

hardly occur in highway bridges when heavily loaded with several trucks. Furthermore, the observations necessary to determine the impact due to accidental obstructions when the bridge is loaded in this fashion are almost impossible.

In addition to the direct results by tests on bridges, valuable information can be deduced from impact tests on pavements such as those conducted by the U. S. Bureau of Public Roads. To enable such results to be utilized, it is necessary to determine, first, the relation between the wheel blow on a pavement, and the blow on a bridge floor, under the same surface conditions; and, second, the actual dynamic stresses produced in the bridge member by a given wheel blow. The relative force of a blow, in percentage of static load, delivered by a truck wheel passing over an obstruction depends on the speed, the height of the obstruction, the flexibility of the tires, the percentage of the unsprung to the total weight, and the flexibility of the pavement upon which the wheel drops. It is less on a concrete bridge floor than on a concrete pavement; and limited data indicate that it may be less on certain types of non-rigid pavement than on concrete. The higher the percentage of unsprung weight, the greater will be the ratio of the blow to the static load; and, within certain limits, the greater the speed, the greater the blow. Furthermore, due to inertia and the time element of deflection and deformation, the stresses produced in the bridge member by the force due to a falling wheel is considerably less than would result from a static load of the same amount as this force. All these factors have been taken into account in the analysis of the data and the formulation of the conclusions of this report.

Definitions

By reason of the numerous quantities and ratios involved in the problem, it is desirable to define carefully some of the terms used.

Dynamic Force.—The dynamic force of a wheel blow is the maximum pressure of a truck wheel upon the pavement or bridge floor when the truck is in motion.

Impact Increment of Dynamic Force.—This increment is the amount by which the dynamic force exceeds the weight of the wheel. It is commonly expressed as a percentage of the static weight.

Impact Increment of Stress.—This is the amount by which the actual stress due to the moving load exceeds the static stress. It is also commonly expressed in terms of percentage of static stress.

Stress Ratio.—Stress ratio is the ratio of the actual dynamic stress produced in a member to the stress that would have occurred if a static load equal in magnitude to the dynamic force were applied at the same place.

Sprung Weight of a Truck.—This weight includes all weight carried by the springs.

Unsprung Weight.—This consists of the remaining weight: Wheels, axles, housings, and springs.

Explanation of the Data

On the Ames project, work was done on five steel bridges with concrete floors (four with steel stringers and one with concrete slab between floor-beams) and on seven steel bridges with timber floors on steel stringers. The individual loads with few exceptions were 3½-ton Liberty trucks, which weighed about 5 tons and were loaded with various amounts up to 10 tons, making a maximum total load of 15 tons. The tires were solid dual rubber, worn down so that their deflection was 0.19 in. under a static load of 10 000 lb. Speeds were attained up to 15 miles per hour. Stress measurements were taken on the stringers, floor-beams, and a few truss members by various extensometers; and the force of the blow of the wheel on the floor was determined (part of the time) by accelerometers.

The U. S. Bureau of Public Roads carried on extensive tests in co-operation with the Society of Automotive Engineers and the Rubber Association of America on impact on pavements from trucks equipped with various kinds of tires. The force of the blows was measured by accelerometers, and the load was passed over obstructions of various heights and at various rates of speed. Impacts were determined on smooth concrete pavement and also on old stone block pavement. Tests were also made by the Bureau of Public Roads on the relative impact on pavements of different kinds, including concrete, brick, and various bituminous pavements.*

With the assistance of Professor E. B. Smith,† an interpretation has been made of the 1921 "copper cylinder" impact data secured, under his direction, by the U. S. Bureau of Public Roads, along lines suggested in his papers‡ descriptive of those results. This interpretation has been made more readily and more clearly by comparison with similar data taken from *Bulletin 75* of the Engineering Experiment Station of Iowa State College.

In order to adapt to the determination of impact on bridges, the mass of data of the Bureau of Public Roads giving impact on pavements, the Engineering Experiment Station of Iowa State College made a few tests during the summer of 1927, in which the force of the blow from the same loads and speeds were determined upon pavements and upon a number of bridges. These latter data have not been previously published, but are included in Appendix D with the consent of the Station.

Appendix A contains a further description of the Ames tests, with abstracts from *Bulletin 75* of the Iowa Engineering Experiment Station. In this appendix are selected diagrams showing the actual results obtained on impact stresses at various speeds and under various conditions; also the impact stress as related to the dynamic force exerted on the bridge floor. The relation between the stress produced by dynamic force and static force of the same amount—the "stress ratio"—is also shown by diagrams. The selected data for impact stresses in trusses show very clearly the relatively small impact where the structure is sufficiently loaded to produce stresses of as much as 5 000 or 6 000 lb.

* *Public Roads*, June, 1926, and August, 1928.

† Of the Division of Tests, U. S. Bureau of Public Roads, until the summer of 1925 and since that time Research Professor of Mechanical Engineering, Iowa State College, Ames, Iowa.

‡ *Public Roads*, March and December, 1921.

per sq. in., and illustrate the difficulty involved in this part of the impact investigation.

Appendix B contains extracts from the published reports of the tests of the U. S. Bureau of Public Roads on pavements. These illustrate very well the effect of variations of conditions.

Appendix C contains results of an analysis by the Committee to render possible the approximate determination of impact stresses in floor members in terms of the separate variables involved, namely, the percentage of unsprung weight of the truck, the height of the obstruction, the speed of the truck, the relative tire deformation, and the stress ratio. It is thought that such an analysis, while having no pretention to great accuracy, will assist engineers in making an independent estimate of impact, particularly those who may undertake future experiments.

Appendix D is an analysis of the relation between impact on a concrete pavement and impact on a concrete bridge floor, and helps to relate the work of the Bureau of Public Roads previously mentioned to the Ames impact studies. It is interesting to note that the ratio of impact on a concrete bridge floor to that on a pavement when the bridge members are stressed to 8 000 lb. per sq. in., or more, does not exceed 80 per cent.

Impact in Floors of Highway Bridges

Concrete Floors.—A brief consideration of the variables involved shows how extremely difficult it is to reach any definite conclusions as to what impact factor should be used in the design of highway bridge floors.

The results obtained by application of the analysis given in the Appendices are illustrated in Table 1. The term, "worn solid tires," represents the hardest worn solid tire that has been noted. For it the value of d (the deflection of the tire due to a static load of 10 000 lb.) = 0.1 in.; for "new solid tires", d = 0.6 in.; and for "pneumatic tires", d = 2.0 in. The "normal load" consists of a truck with live load equal to the rated capacity, for which p (the percentage of unsprung weight to total weight) = 25. "Over load" represents an overloaded truck—for example, one weighing 5 tons carrying a live load of 10 tons, making a total of 15 tons and proportioned so that p = 15.

TABLE 1.—Percentage of Calculated Impact Increments of Stress in Highway Bridge Floors for Trucks.
(Speed, 15 Miles per Hour.)

Obstruction.	Worn Solid Tires.		New Solid Tires.		Pneumatic Tires.	
	Normal load.	Over load.	Normal load.	Over load.	Normal load.	Over load.
None	22	12	8	4	4	2
1 in. by 2 in.	210	102	57	30	30	12
2 in. by 4 in.	475	275	122	58	50	30

The values in Table 1 cover a wide range of conditions, and show what high impacts may be expected in some cases. The Committee does not pro-

pose this table as a suggested standard, but presents it as a reasonably accurate picture of the effect of various factors on the impact on highway bridge floors. It has been computed from Equation (1), Appendix C and the stress ratio curve of Fig. 8, Appendix A.

Timber Floors.—The limited work on timber floors with steel stringers indicates that the best of timber floors will compare favorably with clean concrete floors in regard to impact. It also indicates that on the roughest timber floors investigated the impact was about the same as for 1-in. obstructions on concrete floors.

Recommendation for Impact in Floors of Highway Bridges.—As a basis for its recommendations, the Committee has considered the following facts:

First.—That stresses due to static loads and to impact are important, as regards the safety of the structure, only when they approach design values.

Second.—That the percentage of impact increment decreases as the loads increase, and, therefore, as the unit stresses increase.

Third.—That the larger impacts observed in the tests were produced by obstructions, such as would be accidental and infrequent under actual traffic conditions.

Fourth.—That the actual occurrence, on a bridge, of loads having a magnitude corresponding to those used in the design of modern structures is infrequent.

Fifth.—That the simultaneous occurrence on a bridge floor of a maximum truck load and an accidental obstruction capable of producing high impact will be such a rare coincidence that presumably the factor of safety will usually provide safety for this condition.

This Committee recommends, therefore, that for the design of highway bridge floors and floor-beam suspenders, the impact increment of stress be assumed as 25% of the live load stress. It should be used only when the floors are sufficiently smooth to conform to good modern practice; and unusual conditions should be provided for in accordance with the judgment of the individual designer. The Committee believes that this report contains information, with necessary precision, for guidance in unusual conditions.

Impact in Trusses of Highway Bridges

The published results for impact in the trusses of highway bridges are of interest and value more for the comparison between computed and observed static stresses than for impact. The many experimental results for impact are doubtless correct within a reasonable precision, but they are not representative of the conditions that control in the design of bridges.

The results have been studied in various ways and are reproduced in Fig. 1 for trusses with concrete floors and in Fig. 2 for trusses with timber floors. Evidently, the impact decreases as the unit stresses increase. It is apparent from a practical standpoint that impact is important in design only as the combined stresses due to dead loads, static live loads, and impact approach design stresses. Such stresses have not been reached or closely approached in known experiments.

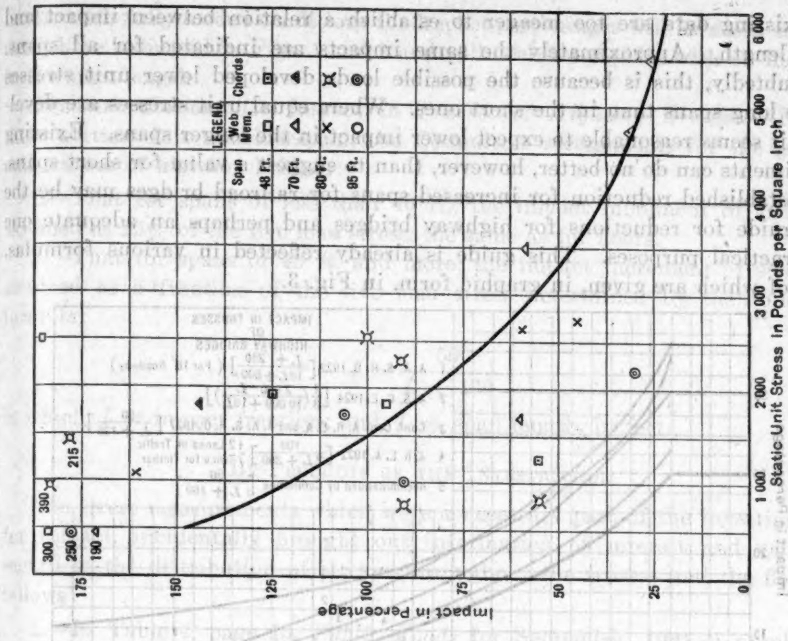

FIG. 2.—IMPACT IN TRUSSES OF HIGHWAY BRIDGES WITH TIMBER FLOORS.

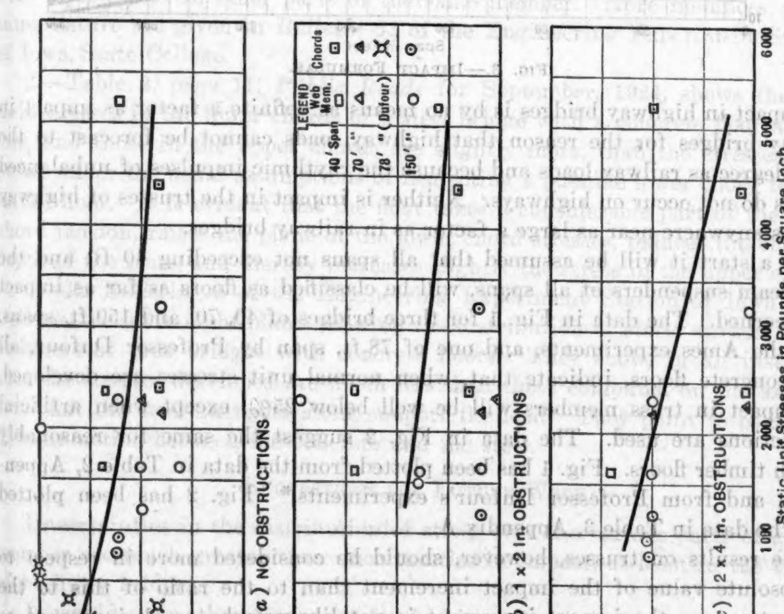

FIG. 1.—IMPACT IN TRUSSES OF HIGHWAY BRIDGES WITH CONCRETE FLOORS.

Existing data are too meager to establish a relation between impact and span length. Approximately the same impacts are indicated for all spans. Undoubtedly, this is because the possible loads developed lower unit stresses in the long spans than in the short ones. Where equal unit stresses are developed, it seems reasonable to expect lower impact in the longer spans. Existing experiments can do no better, however, than to suggest a value for short spans. The established reduction for increased spans for railroad bridges may be the best guide for reductions for highway bridges and perhaps an adequate one for practical purposes. This guide is already reflected in various formulas, some of which are given, in graphic form, in Fig. 3.

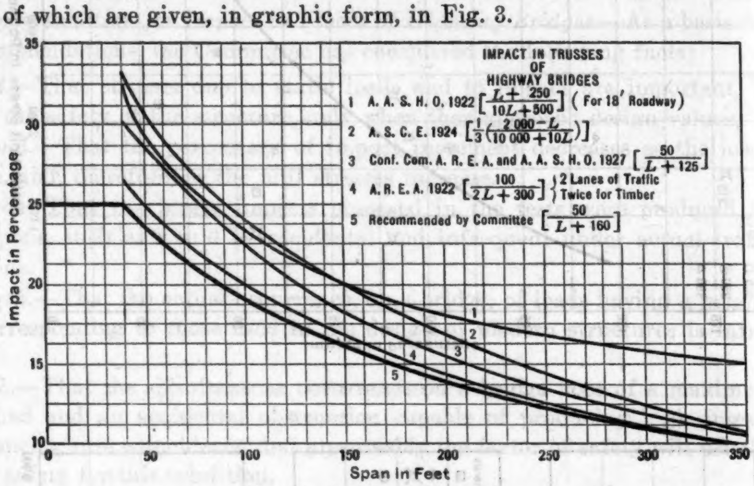

FIG. 3.—IMPACT FORMULAS.

Impact in highway bridges is by no means as definite a factor as impact in railway bridges for the reason that highway loads cannot be forecast to the same degree as railway loads and because the rhythmic impulses of unbalanced drivers do not occur on highways. Neither is impact in the trusses of highway bridges anywhere near as large a factor as in railway bridges.

As a start it will be assumed that all spans not exceeding 40 ft. and the floor-beam suspenders of all spans, will be classified as floors as far as impact is concerned. The data in Fig. 1 for three bridges of 40, 70, and 150-ft. spans, from the Ames experiments, and one of 78-ft. span by Professor Dufour, all with concrete floors, indicate that, when normal unit stresses are developed, the impact in truss members will be well below 25%, except when artificial obstructions are used. The data in Fig. 2 suggest the same for reasonably smooth timber floors. Fig. 1 has been plotted from the data in Table 2, Appendix A, and from Professor Dufour's experiments.* Fig. 2 has been plotted from the data in Table 3, Appendix A.

The results on trusses, however, should be considered more in respect to the absolute value of the impact increment than to the ratio of this to the static stress, as the impact increment is not likely to be much increased as the bridge becomes more fully loaded and the static stresses become greater.

* *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, pp. 1737–1742.

Under such fully loaded conditions the impact percentages will be much smaller than those indicated in Figs. 1 and 2 and will decrease rapidly with an increase in the span length.

Recommendations for Impact in Trusses of Highway Bridges.—Taking these matters into consideration, the Committee recommends for the girders and trusses of highway bridges:

1.—That for spans of less than 40 ft., the impact increment of stress be assumed as 25% of the live load stress, the same as for floors.

2.—That for spans of 40 ft. and more, the impact increment of stress be assumed as a fraction of the live load stress determined by the following formula:

$$I = \frac{50}{L + 160}$$

in which, I = impact fraction; and, L = span length, in feet.

By-Products of the Investigation

The stress measurements which were a necessary part of the investigations for impact, incidentally brought out information of interest and value in regard to the distribution of stresses throughout the trusses and the floor, as follows:

1.—In Table 4, page 13, *Public Roads* for September, 1924, a number of instances are cited where the unit stress in one part of a built member is more than double that in other parts of the same member. More instances of the same nature are given in *Bulletin 63* of the Engineering Experiment Station of Iowa State College.

2.—Table 2, page 11, *Public Roads* for September, 1924, shows that the unit stresses in the lower chords of a steel bridge with a concrete floor are far less, and those in the upper chords are slightly more, than the stresses computed under the usual assumptions of neglecting a possible lower chord tension in the floor. It is evident that the floor takes a considerable part of the lower chord tension, raises the plane of the lower chord stresses, reduces the effective depth of the truss, and thereby increases slightly the stress in the upper chords.

3.—In *Bulletin 75* of the Engineering Experiment Station of Iowa State College, there are eight plates showing the distribution of static stresses in the stringers of four bridges with concrete floors. They show, in all instances, that the stresses developed are much less than those computed on the assumption that the steel stringers alone support the load. They point to consistent T-beam action between the concrete and the steel.

Suggestions for Future Work

Uncertainties in the distribution of stress over the various parts of a truss member are much greater than uncertainties in impact, at least in the trusses of highway bridges.

It would seem, therefore, that further investigations for impact as useful information for a basis for design, should be accompanied, or perhaps preceded, by a general study of the distribution of stresses, including secondary

stresses, stresses due to eccentric connections, etc. Such an investigation should necessarily be of sufficient magnitude to cover a wide range of conditions and to yield conclusions of general application. It would be a worthy companion of the research now under way through the American Society for Testing Materials for studying the distribution of stresses throughout the area of rolled sections. That research is based on the fact that the different parts of a rolled section were really different materials due to unequal working in the process of rolling. Its ultimate effect may be to improve the methods of rolling and to provide material of greater uniformity.

The bridge truss research would give information concerning the degree to which stress is transmitted from one composite part of a riveted (or a welded) member to another. This might possibly lead to better fabrication as well as point out the parts of a structure where the stress distribution is least uniform. In either event, it would contribute to the understanding of some of the factors that must be considered in the determination of unit stresses, a problem which was considered by the Society a few years ago, but regarding which no satisfactory solution was reached.

The bridge truss research should disclose whether there is any need for further work on impact in trusses of highway bridges, and it also might point the way toward the best procedure for the impact investigations if they were found to be desirable.

Respectfully submitted,
Special Committee on Impact in Highway Bridges,
A. H. Fuller, *Chairman*,
Arthur R. Eitzen,
E. F. Kelley,
F. E. Turneaure.

December 5, 1928.

APPENDIX A

Stress and Force Measurements for Impact on Bridges

In order that those who do not have access to *Bulletin 73* of the Engineering Experiment Station of Iowa State College, mentioned in the report, may get a general idea of the manner in which the data of that *Bulletin* were gathered and recorded, a brief description of these factors is given.

The bridges most used were: (*a*) A 150-ft. through riveted steel span, with 6-in. concrete floor on nine lines of 10-in., 25-lb. steel stringers of 18 ft., 9-in. span; (*b*) a 33-ft. I-beam span, with 6-in. concrete floor on nine lines of 15-in., 43-lb. steel stringers; (*c*) a 26-ft. I-beam span, with 8-in. concrete floor on nine lines of 18-in., 54.7-lb. steel stringers; and (*d*) seven light steel spans, with timber floors on steel stringers.

The loads consisted of 3½-ton Liberty trucks with total loads varying from 5 tons for the empty truck to 15 tons, with maximum rear axle load of 12 tons. The unsprung weight of one rear axle was 4 400 lb.

The instruments most used for stress measurements were the McCollum-Peters electric telemeter* and the West extensometer.†

Impact in Floors.—The force of the blows of the truck wheels on the bridge floors was determined by accelerometers which were designed and constructed for the work and described in *Bulletin 75* of the Iowa Engineering Experiment Station. These accelerometers were similar in design and action to those used by the U. S. Bureau of Public Roads.‡

In the preparation of the *Bulletin* from which these conclusions were taken, the stress measurements were platted on a large number of diagrams and median lines were drawn. All these lines were then assembled, for a certain stringer (but without the platted points), on a single diagram, as shown on Fig. 4,§ and other similar diagrams. A study of Fig. 4 brings out certain facts, within the limits of those experiments.

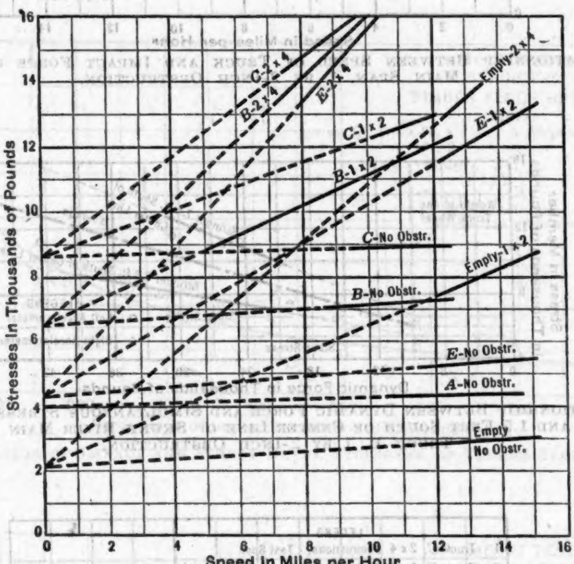

FIG. 4.—RELATIONSHIP BETWEEN SPEED OF TRUCK AND STRESS IN STRINGERS. SIX-INCH CONCRETE FLOORS, SKUNK RIVER MAIN SPAN, STRINGER 7.5 FEET SOUTH OF CENTER LINE.

1.—The increase in stress varies directly with the speed.

2.—The stress increases but slightly with the speed on clean floors (no obstructions).

3.—The stress increases decidedly with the speed when obstructions are used.

4.—The increase in stress, for a given obstruction and speed, is approximately the same for all trucks. This indicates that the impact increment of stress is caused primarily by the unsprung weight of the truck.

* *Bulletin No. 247*, U. S. Bureau of Standards.

† *Proceedings*, Am. Soc. C. E., March, 1923, Papers and Discussions, p. 460.

‡ *Public Roads*, December, 1924, pp. 1-9.

§ *Bulletin 75*, Eng. Experiment Station, Iowa State Coll., Fig. 14.

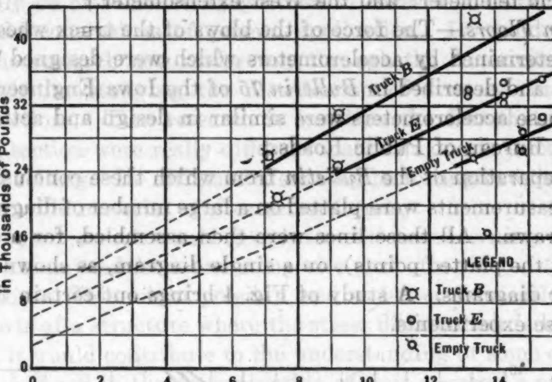

FIG. 5.—RELATIONSHIP BETWEEN SPEED OF TRUCK AND IMPACT FORCE ON SKUNK RIVER MAIN SPAN, 1 BY 2-INCH OBSTRUCTION.

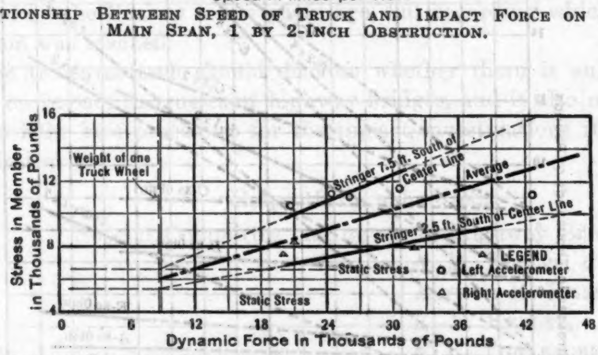

FIG. 6.—RELATIONSHIP BETWEEN DYNAMIC FORCE AND SIMULTANEOUS STRESSES IN STRINGERS 2.5 AND 1.5 FEET SOUTH OF CENTER LINE OF SKUNK RIVER MAIN SPAN, TRUCK B, 1 BY 2-INCH OBSTRUCTION.

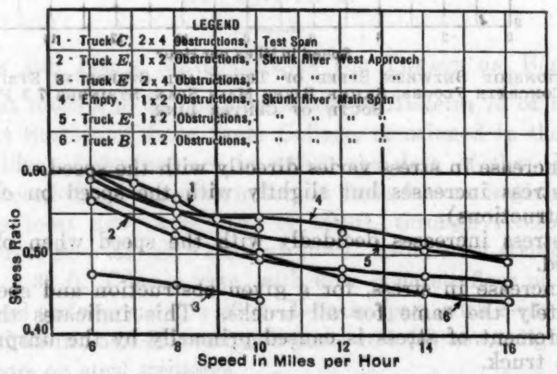

FIG. 7.—RELATIONSHIP BETWEEN SPEED OF TRUCKS AND STRESS RATIO FOR STRINGERS OF CONCRETE FLOOR BRIDGES.

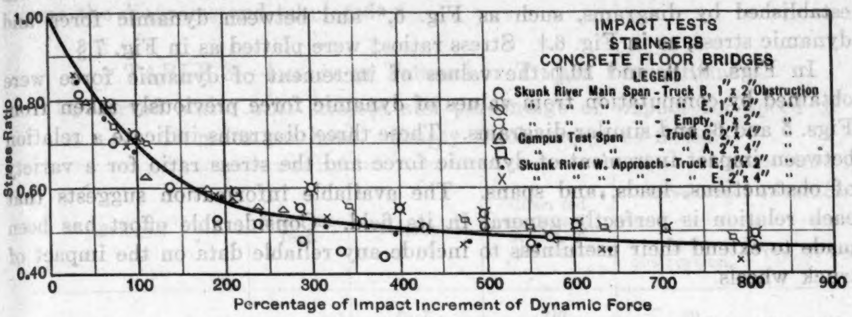

FIG. 8.—RELATION OF IMPACT AND STRESS RATIO, STRINGERS OF CONCRETE FLOOR BRIDGES.

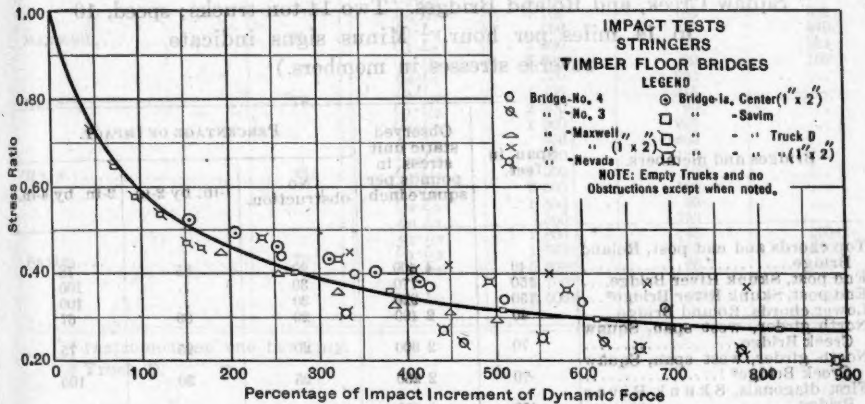

FIG. 9.—RELATION OF IMPACT AND STRESS RATIO, STRINGERS OF TIMBER FLOOR BRIDGES.

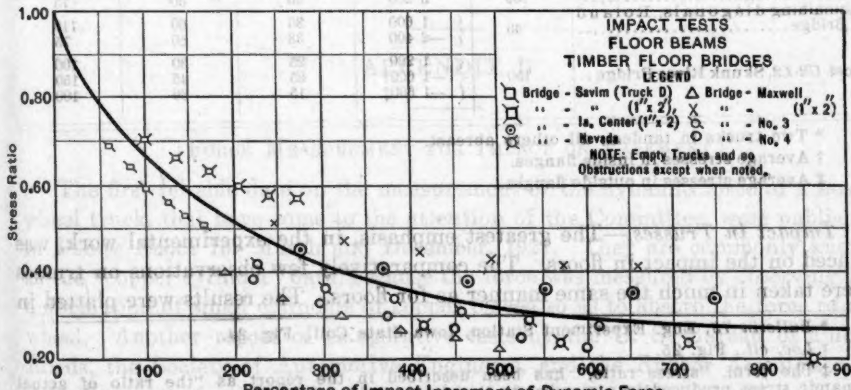

FIG. 10.—RELATION OF IMPACT AND STRESS RATIO, FLOOR-BEAMS OF TIMBER FLOOR BRIDGES.

A relationship between speed and the dynamic force of a wheel blow was established by diagrams, such as Fig. 5,* and between dynamic force and dynamic stress, as in Fig. 6.† Stress ratios‡ were platted as in Fig. 7.§

In Figs. 8, 9, and 10,‖ the values of increment of dynamic force were obtained by computation from values of dynamic force previously taken from Figs. 5 and 6, and similar diagrams. These three diagrams indicate a relation between impact increment of dynamic force and the stress ratio for a variety of obstructions, loads, and spans. The available information suggests that each relation is perfectly general in its field. Considerable effort has been made to extend their usefulness to include any reliable data on the impact of truck wheels.

TABLE 2.—IMPACT IN TRUSSES OF BRIDGES WITH CONCRETE FLOOR.

(Observed static unit stresses and percentage of impact in Skunk River, Squaw Creek, and Roland Bridges. Two 14-ton trucks; speed, 10 to 14 miles per hour. Minus signs indicate reverse stresses in members.)

Bridges and members.	Span, in feet.	Observed static unit stress, in pounds per square inch.	PERCENTAGE OF IMPACT.		
			No obstruction.	1-in. by 2-in.	2-in. by 4-in.
Top chords and end post, Roland Bridge	40	4 400	20	35	78
End post, Skunk River Bridge	150	770	30	..	100
End post, Skunk River Bridge*	150	920	30	..	100
Lower chords, Roland Bridge	40	2 400	20	33	67
North girder, west span, Squaw Creek Bridge	70	2 300	20	25	75
North girder, west span, Squaw Creek Bridge*	70	2 250	25	30	100
First diagonals, Skunk River Bridge	150	2 000	50	..	150
First diagonals, Skunk River Bridge*	150	2 300	30	75	100
First diagonals, Roland Bridge	40	5 200	30	40	100
Remaining diagonals, Skunk River Bridge	150	3 200	20	30	75
Remaining diagonals, Roland Bridge	40	1 600	35	60	110
		−2 400	33	60	75
Post $U2$-$L2$, Skunk River Bridge	150	1 200	25	30	100
		−1 600†	25	45	150
		−1 600‡	15	30	100

* Two trucks in tandem, all others abreast.
† Average stresses in inside flanges.
‡ Average stresses in outside flanges.

Impact in Trusses.—The greatest emphasis, in the experimental work, was placed on the impact in floors. The comparatively few observations on trusses were taken in much the same manner as for floors. The results were platted in

* *Bulletin 75,* Eng. Experiment Station, Iowa State Coll., Fig. 24.

† *Loc. cit.,* Fig. 25.

‡ The term, "stress ratio," has been described in the report as "the ratio of actual dynamic stress produced in a member to the stress that would have occurred if a static load equal in magnitude to the dynamic force were applied at the same place."

§ *Bulletin 75,* Eng. Experiment Station, Iowa State Coll., Fig. 26.

‖ *Loc. cit.,* Figs. 29, 30, and 31.

form similar to those for floors. Percentages of impact increment were taken from the diagrams and tabulated.

TABLE 3.—Impact in Trusses of Timber Floor Bridges.

(Observed static unit stresses and percentage of impact for speed of 10 miles per hour. Empty truck except as noted.)

Bridges.	Span, in feet.	Member.	Observed static unit stress, in pounds.	Percentage of Impact.	
				No obstruction.	1-in. by 2-in.
Nevada	80	L_0-L_1	740	90	...
		U_1-L_1	1 050	160	...
Iowa Center	79	L_0-L_1	2 500*	100	145
		L_1-L_2	850	55	205
		U_1-L_2	2 600	60	...
		U_2-L_2	2 750*	45	...
Maxwell	95	L_0-L_1	1 700	105	315
		L_1-L_2	500	250	540
		U_1-L_1	1 000	90	255
		U_2-L_2	2 200*	30	160
No. 3	79	L_0-L_1	700	390	...
		L_1-L_2	1 400	215	...
		U_1-U_2	2 300	90	...
		U_1-L_3	1 900	225	...
No. 4	48	L_0-L_2	400	300	...
		L_2-L_3	1 900	125	...
		U_1-U_2	1 250	55	...
		U_1-L_2	2 500	185	...
		U_2-L_3	1 850	95	...
Savim	70	L_0 L_1	500	190	...
		L_1-L_2	1 800	145	290*
		U_1-U_2	1 700†	60	130
		U_1-L_1	5 500†	25	50
			4 700†	30	...
		U_2-L_1	3 500†	60	65

* Instrument on one bar only.

† Truck D.

Table 2* gives data for the trusses of bridges with concrete floors, and Table 3,† for trusses of bridges with timber floors. The results from Table 2 have been plotted in Fig. 1 and those from Table 3, in Fig. 2.

APPENDIX B

Force Measurement for Impact on Pavement

The first reliable data on the measurement of the dynamic force of a heavy wheel truck, that have come to the attention of the Committee, were published in *Public Roads* for March and December, 1921. They are commonly known as the "copper cylinder" data, because the force was measured by observing the deformation of small cylinders of copper placed so as to absorb the force of the wheel. Another report of co-operative tests by the U. S. Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America was of great value in checking the results from the co-operative

* Based on Table 4, *Public Roads*, September, 1924, p. 13.

† Based on Table 8, *Bulletin 75*, Eng. Experiment Station, Iowa State Coll.

project at Ames, Iowa, and in interpreting the "copper cylinder" impact data.* Certain diagrams from the report are reproduced as a means of supplementing and interpreting the data which were obtained from other sources.

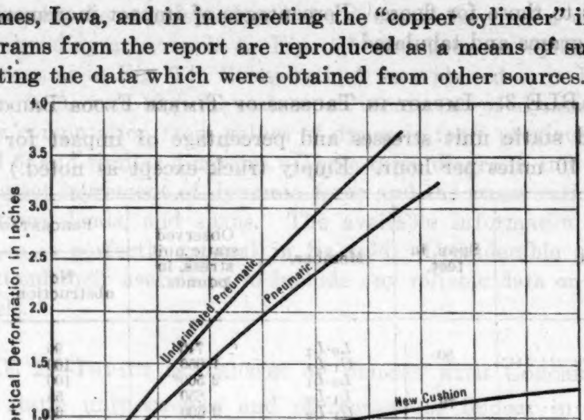

FIG. 11.—TYPICAL CURVES SHOWING TIRE ACTION UNDER STATIC LOAD.

The curves in Fig. 11† give some information concerning the flexibility of tires. The curve for "worn out" solid tires indicates a deformation of about 0.25 in. for a load of 10 000 lb. This is slightly softer than the tires used at Ames, Iowa, which had deformations of about 0.19 in. for the same load.

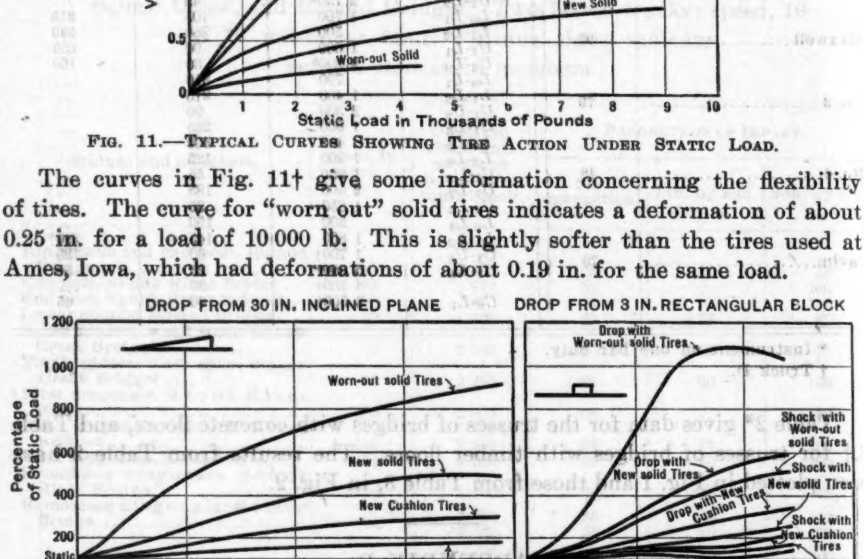

FIG. 12.—EFFECT OF HEIGHT AND TYPE OF OBSTRUCTION ON VERTICAL IMPACT REACTION.

Several results from impact tests are shown in Figs. 12, 13, and 14, all obtained with a 2-ton truck equipped with rated-size dual tires loaded to tire capacity. The speed used in the tests of Figs. 12 and 14 was 12 miles per hour; those of Fig. 13 were variable. Fig. 12‡ shows about the same range of dynamic force of wheel blows as in Figs. 8, 9, and 10, and, at the same time, indicates the variation in the intensity of the blows for different tires and obstructions.

* "Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness," by James A. Buchanan and J. W. Reid, *Public Roads*, June, 1926.

† *Loc. cit.*, Fig. 2.

‡ *Loc. cit.*, Fig. 14.

Fig. 13* shows the relation between the speed and the force of the wheel blow for various tires and obstructions. In comparing these relations with those of *Bulletin 75*, Engineering Experiment Station, Iowa State College, some significant facts are brought out in the form of the curves. A straight line relation between the speed and the blow is indicated in the *Bulletin* for the observed speeds, which were from 5 to 15 miles per hour. The same rela-

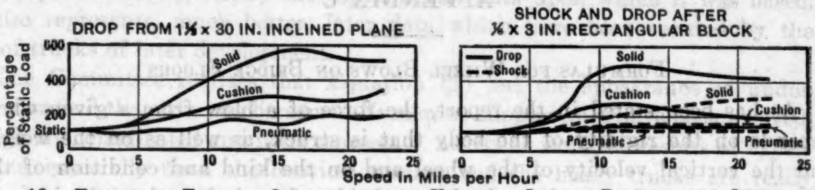

FIG. 13.—EFFECT OF TYPE OF OBSTRUCTION ON VERTICAL IMPACT REACTION AS INFLUENCED BY TRUCK SPEED.

tionship is generally indicated in Fig. 13 for the same speeds, but the latter shows a deviation from a straight line for low speeds (which is not important) and for speeds greater than 15 miles per hour. The data in Fig. 13 make an important contribution in indicating very little, if any, increase in impact for speeds greater than 15 miles per hour. A tendency toward critical speed at about 15 miles per hour was also indicated in one other instance.†

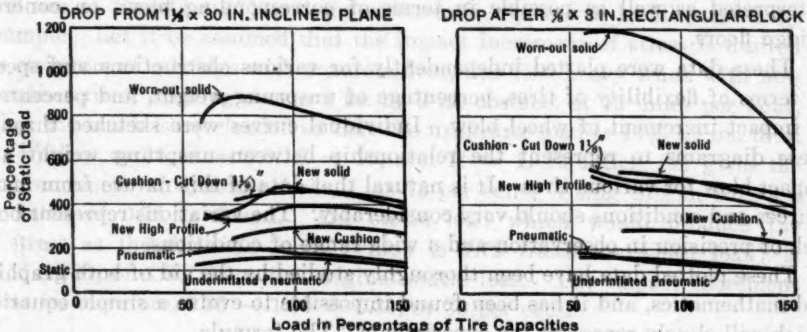

FIG. 14.—EFFECT OF LOAD IN TERMS OF TIRE CAPACITY ON VERTICAL IMPACT REACTION WITH SEVERAL TYPES OF TIRES AND IMPACT.

In Fig. 14‡ is indicated a decreased percentage of impact increment for increased tire loads for various types of tires. This factor is typical of two others mentioned elsewhere in this report: (*a*) That the percentage of impact decreases with an increase in the sprung weight; and (*b*) that the percentage of impact decreases with the flexibility of a structure, or as the unit stresses increase. These two factors suggest that the danger to a structure from an overload on a rubber-tired truck may not be as great as observed values would indicate; also, that for normally stressed and slightly overstressed structures, the impact factors will be less than those observed in the known experiments on bridges.

* "Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness," by James A. Buchanan and J. W. Reid, *Public Roads*, June, 1926, Fig. 15.

† *Public Roads*, August, 1928, Figs. 10, 12, and 13, p. 122.

‡ "Motor Truck Impact as Affected by Tires, Other Truck Factors, and Road Roughness," by James A. Buchanan and J. W. Reid, *Public Roads*, June, 1926, Fig. 16.

Other interesting and pertinent factors were brought out—that impact is perceptibly greater when dual, rather than single, tires are used and decidedly less when the rear load is carried on two axles (four wheels) rather than on the usual arrangement of one axle.

APPENDIX C

Formulas for Wheel Blows on Bridge Floors

As has been stated in the report, the force of a blow from a given wheel depends on the rigidity of the body that is struck, as well as on the weight and the vertical velocity of the wheel and on the kind and condition of the tire. The blow of a moving wheel on a bridge floor is less than the blow of the same wheel on a pavement, whenever the flexibility of the structure is greater than that of the pavement. This condition exists in most bridges, especially where the load is sufficiently great to produce deflections and unit stresses corresponding to fully loaded conditions.

The data from the experiments of the U. S. Bureau of Public Roads on pavements have been compared with corresponding data on the more flexible bridge floors. With this comparison as a basis, all available data have been interpreted as well as possible in terms of corresponding blows on concrete bridge floors.

These data were platted independently for various obstructions and speeds in terms of flexibility of tires, percentage of unsprung weight, and percentage of impact increment of wheel blow. Individual curves were sketched through these diagrams to represent the relationship between unsprung weight and impact blow for various tires. It is natural that data of this nature from many sources and conditions should vary considerably. The variations represent both lack of precision in observation and a wide range of conditions.

These plotted data have been thoroughly studied by the aid of both graphics and mathematics, and it has been found impossible to evolve a simple equation which will closely represent the experiments. The formula,

$$I = \left(\frac{p\, H^{0.45}\, S^{0.54}}{15\, d^{0.3}}\right)^{a} \qquad (1)$$

does represent them and is presented as an equation which best sums up the available knowledge of the impact of wheel blows of present-day heavy trucks on the surface of concrete floors which rest on steel stringers. In this equation:

I = impact increment of dynamic force of wheel blow expressed as a percentage of the static load.

p = unsprung weight expressed as a percentage of total weight of truck.

a = numerical exponent having the value, $\dfrac{1}{0.22\, p^{0.2}}$.

H = height, in inches, of any obstruction on the bridge floor. (Equals about 0.1 in. for the bare concrete floors and an average of the best timber floors for which data are available.)

S = speed of truck, in miles per hour.

d = tire deformation, in inches, under a static load of 10 000 lb.

A simpler equation,

$$I = \frac{1.8\, H \,.\, S \,.\, p^{0.625}}{d^{0.45}}$$

was presented in the report of the Committee for 1926.* Equation (1), however, represents more closely the experimental data upon which it was based. It also represents, much better, later data which have been secured by the use of trucks of later development.

The Committee realizes that Equation (1) has the appearance of undue precision and that it is cumbersome. Numerous simpler ones have been evolved, but all are complicated and none represents, nearly as well, the results of experiments, especially upon the heaviest trucks; and the heavy trucks are really the only ones that are important in design.

Equation (1) gives directly, not the stress impact, but the impact of the blow of the truck wheel on a concrete bridge floor. In order to transform the effect of the blow into stress impact, it is necessary to make use of the stress-ratio curve for concrete floor in Appendix A (Fig. (8)). Equation (1) and the stress-ratio curve have been used in computing Table 1. It is believed they may be safely used for any conditions, within the limits of Table 1 and extended to include percentages of unsprung weight as high as 40.

The use of Equation (1) and the stress-ratio curve will be illustrated by an example. Let it be assumed that the impact increment of stress is desired in the stringers of a bridge with concrete floor, due to a heavy truck with new solid rubber tires, running over a 1 in. by 2-in. obstacle at 15 miles per hour, and that the truck is overloaded so that the unsprung load is 15% of the total load. With $p = 15$, $d = 0.6$, $S = 15$, and $H = 1$, Equation (1) gives the impact increment of dynamic force, I, as 72 per cent. Using this in Fig. 8 a stress ratio of 0.76 is found. The static load which would produce the same stress as the dynamic force (which is 1.72 times the wheel load) is (0.76×1.72), or 1.30 times the original static load. The impact increment of stress is, therefore, $1.30 - 1.00 = 0.30$, or 30 per cent.

APPENDIX D

Relation Between Blow upon Pavement and upon a Bridge Floor

Many data are available, mostly through the U. S. Bureau of Public Roads, which give the intensity of a wheel blow, and, therefore, the impact, on pavements. This information may be made available for determining the impact on bridges by establishing a relationship between the intensity of blows on pavement and on bridges under conditions which otherwise are similar.

A number of experiments not only verify the well-known fact that the intensity of the blow from any wheel is dependent on the flexibility of the surface on which the blow is delivered, but suggest a value for the relationship, fairly well in keeping with the variation in road surfaces, tires, etc.

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* *Proceedings*, Am. Soc. C. E., March, 1926, Papers and Discussions, p. 448.

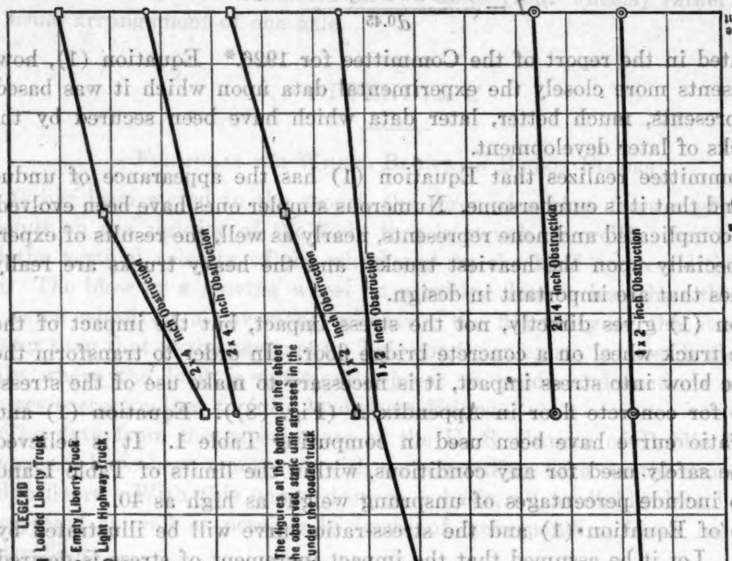

FIG. 16.—COMPARISON OF DYNAMIC FORCE OF WHEEL BLOWS ON PAVEMENTS AND ON BRIDGES.

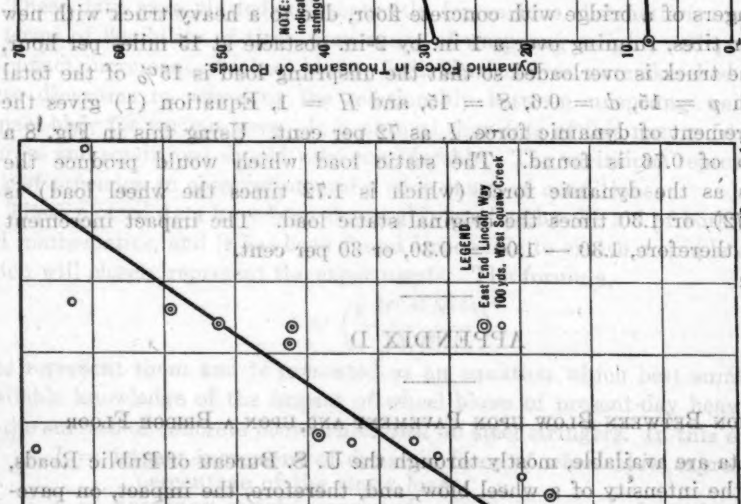

FIG. 15.—DYNAMIC FORCE OF WHEEL BLOWS ON CONCRETE PAVEMENT, EMPTY LIBERTY TRUCK, RIGHT ACCELEROMETER, 1-INCH BY 2-INCH OBSTRUCTION.

Unit stresses, deflection, and impact in the stringers of three bridges of various degrees of flexibility, are given in Table 6 of *Bulletin 75* of the Iowa Engineering Experiment Station. Although these data do not justify a curve or an equation, they do indicate much greater impact for the stiffer structures with small unit stresses and small deflection.

In the summer of 1927, a limited number of runs were made by the Engineering Experiment Station at Iowa State College in which the force of a wheel blow on pavements and on bridges was determined. One of the 3½-ton Liberty trucks described in *Bulletin 75* was used, both empty and loaded. A lighter truck with softer tires was also used. More than thirty diagrams were plotted showing the force of wheel blows in terms of speed. One of these is reproduced as Fig. 15.

TABLE 4.—DYNAMIC FORCE OF WHEEL BLOWS, IN THOUSANDS OF POUNDS, ON PAVEMENT AND ON BRIDGES.

(Speed, 12 miles per hour.)

	Obstruction.	Liberty, Loaded. Accelerometer.			Liberty, Empty. Accelerometer.			Light Highway Truck. Accelerometer.		
		Left.	Right.	Average.	Left.	Right.	Average.	Left.	Right.	Average.
Concrete pavement	1-in.	48.0	......	......	37.2	38.0	37.6	......	......	9.8
Skunk River main span	1-in.	......	......	......	28.5	28.5	28.5	......	......	8.3
Skunk River approach	1-in.	36.0	36.0	36.0	34.0	34.4	34.2	......	......	8.8
Campus test	1-in.	42.4	44.0	43.2	......	......	......	......	......	......
Concrete pavement	2-in.	64.0	65.2	64.6	57.0	56.0	56.5	......	......	18.3
Skunk River approach	2-in.	51.0	......	......	47.8	48.4	48.1	......	......	16.2
Campus test	2-in.	61.0	60.8	60.9	......	......	......	......	......	......
Static wheel load, in pounds	.....		8 800			3 350			1 350	
Unsprung wheel load, in pounds	.....		2 200			2 200			1 000	

Table 4 has been made by reading the wheel blows from these diagrams at speeds of 12 miles per hour. Fig. 16, which has been plotted from Table 4, shows a decided variation of wheel blows on different structures and gives the basis for the degree of variation. Fig. 17 shows percentage effects of blows on a bridge to blows on pavement taken from Fig. 16 and Table 4, plotted against unit stresses in the stringers for the three structures for which observed unit stresses are available. Although the points are small in number and scattering, they give rather definite suggestion that percentages greater than 80 are for such small unit stresses that they would not be significant for design purposes. The necessary computations for plotting Fig. 17 are given in Table 5.

It is impossible to draw precise conclusions from data such as these; neither would precise conclusions be warranted from any experimental or theoretical studies. The inherent variations in structures and vehicles preclude results of high precision. On the other hand, it seems apparent:

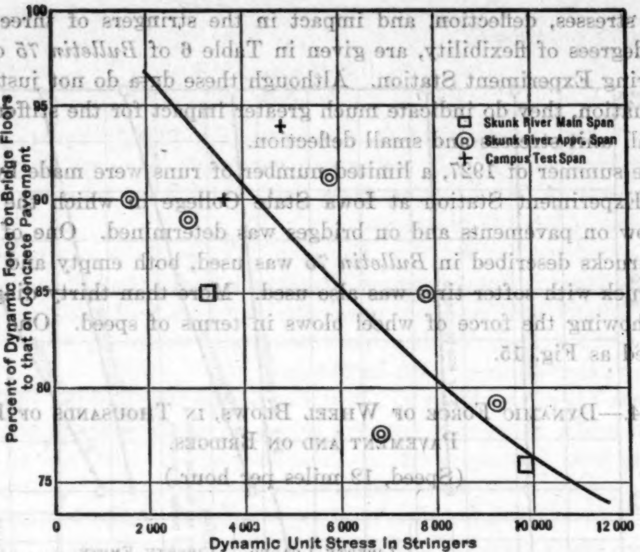

FIG. 17.—RELATION BETWEEN DYNAMIC FORCE ON CONCRETE FLOOR OF BRIDGES AND ON CONCRETE PAVEMENT IN TERMS OF DYNAMIC UNIT STRESS IN STRINGER.

TABLE 5.—DYNAMIC UNIT STRESSES AND RATIO OF WHEEL BLOWS ON THREE BRIDGES WITH CONCRETE FLOOR AND ON CONCRETE PAVEMENT.

(Data from Table 4; speed, 12 miles per hour; loads and forces, in thousands of pounds.)

Static load.	Obstruction.	Dynamic force.	Percentage impact.	Stress ratio.	Equivalent static force.	Static unit stress.	Dynamic unit stress.	Blow on pavement.	Ratio, Blow on bridge / Blow on pavement.
SKUNK RIVER APPROACH SPAN.									
8.8	1-in.	36.0	310	0.52	18.7	3.2	6.8	48.0	0.75
8.8	2-in.	51.0	480	0.49	25.0	3.2	9.1	64.6	0.79
3.35	1-in.	34.2	920	0.47	16.0	1.2	5.8	37.6	0.91
3.35	2-in.	48.1	1 340	0.45	21.6	1.2	7.8	56.5	0.85
1.35	1-in.	8.8	570	0.48	4.2	0.5	1.6	9.8	0.90
1.35	2-in.	16.2	1 100	0.46	7.5	0.5	2.8	18.3	0.89
SKUNK RIVER MAIN SPAN.									
3.35	1-in.	28.5	750	0.48	13.7	2.4	9.8	37.6	0.76
1.35	1-in.	8.3	510	0.49	4.1	1.0	3.0	9.8	0.85
CAMPUS TEST SPAN.									
8.8	1-in.	48.2	400	0.51	22.0	1.5	3.7	48.0	0.90
8.8	2-in.	60.9	600	0.48	29.0	1.5	4.9	64.6	0.94

First.—That the impact is less on a bridge than on the rigid types of pavements; and,

Second.—That the intensity of a blow from a truck wheel on a bridge floor, when the unit stresses in stringers or floor-beams approximate design values, may approach, but will not exceed, 80% of the blow of the same wheel on a reasonably rigid pavement.

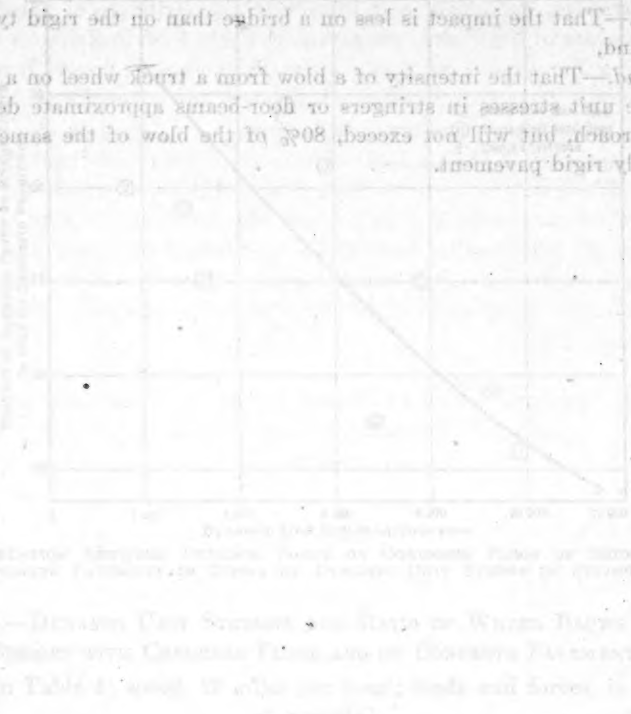

TABLE 5.—Dynamic Unit Stresses and Ratio of Wheel Blows on T[illegible] Bridges with C[illegible] Floors and on Concrete Pavement.

([illegible])

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USE OF WATER ON FEDERAL IRRIGATION PROJECTS

By E. B. Debler,* M. Am. Soc. C. E.

Synopsis

From the initiation of each of the irrigation projects of the United States Bureau of Reclamation it has been customary to ascertain and record the use of water. Operation and maintenance charges have always been based on the quantity of water delivered to individual farms, a method which was made mandatory by Act of Congress approved August 13, 1914. Other justifications for the measurement of water have been: The desirability of securing data for use in designing distribution systems; the necessity for obtaining data to use in determining project areas; and the desire to foster the most economical use of the available irrigation supply.

The results of field measurements of water delivered to farms are assembled annually in the project offices and are summarized in their respective project histories.

The project histories, together with supplemental data otherwise obtained, constitute the source of information from which the data incorporated in this paper have been compiled. In abstracting the results of the field measurements many of the original compilations were revised so that comparable data for all projects could be presented herein.

In tabular form are given the quantities of water delivered to the farms on the various projects each month during the different years of operation, the data being arranged chronologically and one table being devoted to each project or each important sub-division thereof. The tables also include pertinent related data, such as total irrigated areas, total mileage of canals and laterals operated, precipitation during the irrigation season, mean air temperatures, percentages of irrigated areas in different crops, canal and lateral losses, waste, etc. A summary gives the average use of water on the various projects during the 10-year period from 1917 to 1926, inclusive. Pertinent related data and information are also shown, as in the more detailed tabulations.

It is believed that the data included in the tables will be found useful to most engineers engaged in irrigation work. The results of the field measure-

Note.—The subject of this paper is one of several selected by the Special Committee on Irrigation Hydraulics for study and research. The paper was prepared for the consideration of the Committee in order to make available for general use much valuable data heretofore found only in the files of the United States Bureau of Reclamation. Written discussion on this paper will be closed in August, 1929.

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ments of use of water cover a wide range in climatic and soil conditions as well as several years of project operation under varying stages of development. The tabulated data do not possess a high degree of scientific accuracy. Nevertheless, they are believed to be sufficiently accurate for the purposes for which they were obtained.

From the hitherto unpublished records of the U. S. Bureau of Reclamation, considerable data have been assembled and tabulated, to show the variation in use of irrigation water. The many projects included are indicated on Fig. 1, a map of the western part of the United States, showing the locations of the Federal irrigation projects. Numbers enclosed in circles correspond to the numbers of the tables containing the detailed data on the use of water.

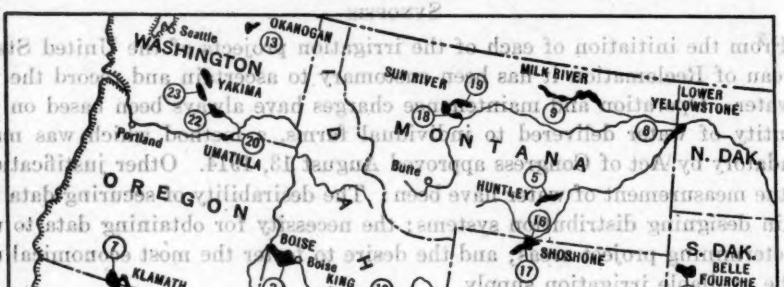

FIG. 1.

Compilations for these twenty-four projects are given in Tables 1 to 25, inclusive, corresponding. The data selected are those considered most useful to an engineer in designing or reviewing an irrigation plan. In general, the information is self-explanatory and hence many of the tables require no comment. However, a few remarks may well be given dealing with the particular aspects of the general problem and of this special investigation.

TABLE 1.—USE OF WATER ON THE BELLE FOURCHE IRRIGATION PROJECT OF SOUTH DAKOTA.

(Average elevation, 2 800 ft.; in clay and sandy soil; 58 miles concrete-lined.)

Calendar year.	Irrigated area, total for year.[2]	Area commanded by constructed canal system.[8]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[3] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April, preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay and pasture.	Small grain.	Furrow crops.	Percentage of Total Diversions for Season. Canal and lateral losses.[4]	Waste.	Delivered to farms.[7]
1912	27 897	65 852	448	......	......	......	......	0.00	0.68	0.27	0.03	0.11	......	......	......	1.09	1.10	2.19	0.50	63	25	61	14	32	16	52
1913	32 881	65 868	462	......	......	......	......	0.02	0.39	0.69	0.23	0.11	......	......	......	1.40	0.76	2.20	0.25	66	31	59	10	51[5]	[5]	49
1914	37 454	68 852	498	......	......	......	......	0.02	0.48	0.57	0.24	0.14	......	......	......	1.45	0.78	2.23	0.45	67	43	43	14	44	7	49
1915	44 067	78 591	528	......	......	......	......	0.01	0.02	0.10	0.13	0.09	0.02	......	......	0.37	1.49	1.86	0.56	60	52	32	16	31	35	34
1916	48 468	78 567	529	......	......	......	......	0.01	0.09	0.47	0.10	0.13	0.01	......	......	0.81	0.86	1.67	0.32	61	59	31	10	16[6]	59	25
1917	50 272	83 835	612	......	......	......	......	0.00	0.19	0.57	0.30	0.15	......	......	......	1.21	0.49	1.70	0.47	63	60	27	18	31	15	54
1918	52 445	82 592	612	......	......	......	......	0.03	0.46	0.17	0.26	0.07	......	......	......	0.99	1.04	2.03	0.48	66	60	30	10	34	18	48
1919	56 638	82 797	615	......	......	......	......	0.18	0.51	0.45	0.23	0.09	......	......	......	1.46	0.33	1.79	0.56	67	63	28	9	35	11	54
1920	43 022	82 655	615	......	......	......	......	0.00	0.00	0.14	0.48	0.13	......	......	......	0.75	1.39	2.14	0.74	67	63	28	9	42	6	52
1921	55 100	82 363	615	......	......	......	0.01	0.16	0.29	0.44	0.26	0.13	......	......	......	1.29	0.63	1.92	0.29	67	64	23	13	38	11	51
1922	31 150	82 194	545	......	......	......	......	0.01	0.01	0.08	0.50	0.31	......	......	......	0.91	1.19	2.10	0.42	66	65	21	14	37	16	47
1923	30 552	81 843	506	......	......	......	......	0.06	0.14	0.36	0.07	0.10	......	......	......	0.73	1.68	2.41	0.40	65	67	14	19	39	26	35
1924	48 400	81 897	455	......	......	......	......	0.02	0.38	0.37	0.30	0.13	......	......	......	1.20	0.46	1.66	0.48	62	67	9	23	38	19	43
1925	48 800	81 816	455	......	......	......	......	0.12	0.16	0.52	0.38	0.12	......	......	......	1.30	0.57	1.87	0.64	65	59	17	24	42	18	45
1926	36 260	74 569	443	......	......	......	......	0.05	0.11	0.48	0.39	0.15	......	......	......	1.18	0.87	2.05	0.34	60	54	25	21	39	23	38

[1] Climatological records, mean of Orman and Vale, S. Dak.

[2] 1912-19, total cropped area; irrigated area considerably less in years of high rainfall.

[3] Project charges based on these deliveries; actual deliveries estimated as 15% greater.

[4] Actual losses estimated as averaging 7% less than shown, due to under-measurement of deliveries.

[5] Waste not measured, but included in loss.

[6] 49% of total diversions wasted in order to lower reservoir water surface for dam repairs.

[7] Actual deliveries estimated as averaging 7% more than shown, due to under-measurement.

[8] Irrigable area originally estimated at 90 000 acres; reduction due to suspension of construction and to elimination of poor lands.

TABLE 2.—USE OF WATER ON THE BOISE IRRIGATION PROJECT OF IDAHO.

(Average elevation, 2 500 ft.; chiefly in volcanic ash and sandy loam soil; 37 miles of pipe, flume, and concrete-lined canals.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[4] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[2]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[2]	Mean temperature, April to October, in degrees Fahrenheit.[2]	Percentage of Area in Different Crops.[1] Alfalfa, hay and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season.[5] Canal and lateral losses.[6]	Waste.[7]	Delivered to farm.[8]
1912	61 725	120 000	966	......	......	......	0.02	0.26	0.55	0.61	0.26	0.17	0.02	......	[3]	1.89	0.77	2.66	0.58	59	48	39	7	6	51	4	45
1913	76 265	127 000	969	......	......	......	0.04	0.57	0.72	0.54	0.36	0.12	0.03	......	[3]	2.38	0.78	3.16	0.30	60	48	38	9	5	....	....	....
1914	83 714	127 000	970	......	......	......	0.29	0.70	0.57	0.61	0.27	0.12	0.06	......	[3]	2.62	0.44	3.06	0.43	62	52	28	15	5	40	4	56
1915	97 127	146 190	979	......	......	......	0.37	0.19	0.73	0.68	0.55	0.25	0.04	......	......	2.81	0.40	3.21	0.29	61	47	36	13	4	37	2	61
1916	101 315	143 810	981	......	......	......	0.26	0.69	0.62	0.89	0.64	0.39	0.07	......	......	3.56	0.32	3.88	0.49	60	55	34	7	4	35	6	59
1917	116 686	143 780	988	......	......	......	....	0.37	0.81	0.76	0.64	0.47	0.02	......	......	3.07	0.36	3.43	0.35	61	55	34	6	5	36	3	61
1918	131 500	163 938	989	......	......	......	0.15	0.86	0.83	0.90	0.65	0.34	0.02	......	......	3.75	0.39	4.14	0.48	63	54	36	7	3	28	6	66
1919	146 500	165 305	989	......	......	......	0.12	0.89	0.92	0.89	0.50	0.02	....	......	......	3.34	0.16	3.50	0.38	62	55	32	10	3	29	1	70
1920	150 760	165 841	1 002	......	......	......	0.11	0.80	0.85	0.95	0.77	0.10	0.02	......	......	3.60	0.43	4.03	0.42	61	61	26	11	2	26	1	73
1921	153 000	167 483	1 016	......	......	......	0.25	0.53	0.79	0.90	0.75	0.33	0.09	......	......	3.64	0.41	4.05	0.55	62	57	30	11	2	26	1	74
1922	155 000	167 555	1 016	......	.....	......	0.04	0.71	0.81	0.91	0.70	0.26	0.04	......	......	3.47	0.28	3.75	0.40	62	48	30	20	2	29	....	71
1923	155 500	167 655	1 019	......	......	......	0.37	0.73	0.43	0.90	0.77	0.45	0.05	......	......	3.70	0.65	4.35	0.29	61	49	29	19	3	32	....	68
1924	156 000	167 754	1 019	(Severe shortage)			0.18	0.72	0.50	0.25	0.04	0.02	0.01	0.05	......	1.77	0.18	1.95	0.25	62	61	22	14	3	36	....	64
1925	156 000	167 782	1 019	......	......	......	0.27	0.93	0.78	0.88	0.91	0.33	0.05	......	......	4.15	0.40	4.55	0.55	62	47	33	18	2	21	....	79
1926	158 000	167 780	1 019	(Heavy shortage)			0.26	0.76	0.62	0.45	0.24	0.09	0.01	......	......	2.43	0.29	2.72	0.44	64	58	31	8	3	32	....	68

1 Based on crop census for an average of 75% of irrigated area.

2 Climatological records at Caldwell, Idaho.

3 Arrowrock Reservoir under construction, placed in operation in 1915, with supply to lands above Deerflat Reservoir limited prior thereto.

4 Project charges based on these deliveries; actual deliveries estimated as 5% greater.

5 Total diversions from Boise River at Boise Diversion Dam above Boise, Idaho, plus outflow from Deerflat Reservoir minus inflow to Deerflat Reservoir except for 1912 when there was other inflow to the reservoir; Deerflat Reservoir is served by main canal.

6 Actual losses estimated as averaging 3% less than shown due to under-measurement of deliveries.

7 Includes only wasteways where records were kept. No records after 1922. Waters wasted from upper system but caught in Deerflat Reservoir not charged as waste.

8 Actual deliveries estimated as averaging 3% more than shown due to under-measurement.

9 Final irrigable area 167 780 acres (under Government canals, with water supply from Boise River).

TABLE 3.—USE OF WATER ON THE CARLSBAD IRRIGATION PROJECT OF NEW MEXICO.

(Average elevation, 3 100 ft.; in sandy loam and gravel; 25% of system concrete-lined.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canal system.[6]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[2]													Precipitation, February to November, in feet.[1]	Water delivered plus precipitation, February to November, in feet.	Precipitation, December to January preceding, in feet.[1]	Mean temperature, February to November, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.[3]		
				January.[7]	February.[7]	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay and pasture.	Small grains.	Furrow crops.[8]	Trees.	Canal and lateral losses.[4]	Waste.[9]	Delivered to farms.[5]
1912	13 459	20 277	45	......	......	0.12	0.45	0.44	0.48	0.44	0.62	0.18	0.10	0.05	......	2.88	0.99	3.87	0.08	65	63	....	35	2	48	6	46
1913	14 260	20 261	45	......	......	0.06	0.49	0.43	0.17	0.44	0.40	0.21	0.06	0.06	......	2.32	1.20	3.52	0.08	66	70	1	26	3	54	7	39
1914	12 690	20 261	45	......	0.05	0.05	0.52	0.32	0.17	0.50	0.55	0.26	0.02	......	......	2.44	1.31	3.75	0.07	67	71	....	27	2	56	9	35
1915	13 470	24 796	45	......	......	......	0.06	0.32	0.58	0.43	0.35	0.21	0.15	0.04	......	2.14	1.46	3.60	0.30	66	54	15	29	2	49	15	36
1916	16 600	24 775	45	0.01	0.15	0.22	0.39	0.51	0.50	0.30	0.02	0.16	0.06	0.11	......	2.43	1.62	4.05	0.07	67	54	19	26	1	58	3	39
1917	16 882	24 775	45	0.03	0.02	0.15	0.53	0.42	0.21	0.08	0.40	0.21	0.19	0.10	......	2.34	0.46	2.80	0.03	66	53	13	34	...	52	1	47
1918	19 460	24 775	45	......	0.11	0.12	0.50	0.38	0.39	0.14	0.32	0.38	0.02	0.07	......	2.43	0.57	3.00	0.05	65	50	8	42	...	50	1	49
1919	20 363	24 991	45	......	0.02	......	0.43	0.37	0.41	0.46	0.49	0.21	0.01	......	......	2.40	1.56	3.96	0.05	65	44	3	53	...	53	4	43
1920	22 172	24 991	45	......	......	0.09	0.62	0 31	0.16	0.56	0.32	0.32	0.04		......	2.42	1.13	3.55	0.11	65	32	2	66	...	52	8	40
1921	23 814	24 991	45	......	0.10	0.17	0.52	0.21	0.09	0.47	0.51	0.27	0.12	0.03	....	2.49	0.77	3.26	0.07	67	38	5	57	...	42	15	43
1922	24 078	24 991	45	0.10	0.03	0.14	0 42	0.23	0 31	0.51	0.44	0.08	0.10	......	......	2.36	0.91	3.27	0.02	65	33	2	65	...	42	9	49
1923	24 063	24 991	45	0.13	0.02	0.01	0.55	0.39	0.35	0.41	0.36	0.14	0.02	......	......	2.38	1.02	3.40	0.04	65	25	1	74	...	47	5	48
1924	24 460	25 045	45	......	0.01	0.03	0.57	0.32	0.47	0.52	0.63	0.12	0.04	......	......	2.71	0.21	2.92	0.18	66	20	....	80	...	45	5	50
1925	24 778	25 045	45	0.12	0.14	0.10	0.43	0.11	0.07	0.17	0.51	0.13	0.06	......	......	1.84	0.80	2.64	0.04	68	19	....	81	...	49	8	43
1926	25 278	25 056	45	0.04	0.18	0.10	0.48	0.12	0.37	0.40	0.43	0.13	0.00	......	.....	2.25	1.12	3.87	0.06	66	20	1	79	...	51	6	43

[1] Climatological records at Carlsbad, N. Mex.

[2] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[3] Total diversion at Avalon is estimated by adding 5% to sum of measured discharges at Main Canal flume over Pecos River and East Canal Head-Gate. Diversions from Black River also included.

[4] Actual losses estimated as averaging 4% less than shown, due to under-measurement of deliveries.

[5] Actual deliveries estimated as averaging 4% more than shown, due to under-measurement.

[6] The project that may permanently be irrigated is limited by water supply and in years of abundant run-off may be exceeded as additional arable lands are commanded by the existing distribution system. The final irrigable area is 25 000 acres.

[7] Irrigation in these months is largely done in anticipation of shortages later in season and with waters that would otherwise be wasted.

[8] Largely cotton.

[9] Indicated waste is largely at ends of season and of waters which can not be conserved.

TABLE 4.—USE OF WATER ON THE GRAND VALLEY IRRIGATION PROJECT OF COLORADO.

(Average elevation, 4 700 ft.; much of system specially puddled-lined; main canal largely in highly pervious strata; many of laterals in sand, with porous subsoil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.[9]	Delivered to Farms, in Hundredths of Acre-Feet per Acre. January.[2]	February.[3]	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees, Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Percentage of Total Diversions for Season.[8] Canal and lateral losses.[5,7]	Waste.[6]	Delivered to farm.[8]
1916	1 742	15 000	80	......	......	......	......	0.46	0.74	0.62	0.21	0.27	0.08	0.04	......	2.42	0.57	2.99	0.85	64	14	39	14	33	54	32	14
1917	5 289	30 500	137	......	......	......	0.06	0.25	0.70	0.97	0.72	0.38	0.22	0.21	0.03	3.54	0.39	3.93	0.15	64	20	30	35	15	44	23	33
1918	8 102	30 500	168	......	......	......	0.13	0.75	0.83	0.66	0.53	0.39	0.27	0.12	......	3.68	0.39	4.07	0.24	66	23	27	40	10	43	17	40
1919	10 049	30 500	175	......	......	......	0.10	0.63	0.95	0.82	0.68	0.42	0.21	......	......	3.81	0.35	4.16	0.24	67	29	25	42	4	44	17	39
1920	11 734	30 500	187	......	......	......	......	0.46	0.68	0.69	0.63	0.33	0.28	......	......	3.07	0.45	3.52	0.45	64	38	25	34	3	49	18	33
1921	12 290	30 500	200	......	......	......	0.22	0.76	0.78	0.84	0.42	0.33	0.28	......	......	3.58	0.72	4.30	0.16	65	41	25	32	2	38	22	40
1922	12 372	30 500	212	......	......	......	0.09	0.77	0.88	0.85	0.59	0.36	0.20	......	......	3.74	0.33	4.07	0.25	66	41	21	36	2	44	22	34
1923	12 870	30 500	212	......	......	......	0.16	0.79	0.75	0.82	0.70	0.42	0.13	......	......	3.77	0.44	4.21	0.32	64	49	11	38	2	42	24	34
1924	13 456	30 000	214	......	......	......	0.25	0.96	0.68	0.97	0.92	0.38	0.18	......	......	4.34	0.54	4.88	0.27	65	49	21	27	3	39	20	41
1925	13 488	30 350	214	......	......	......	0.35	1.11	0.89	0.82	0.73	0.24	0.07	......	......	4.21	0.61	4.82	0.31	66	51	12	36	1	35	22	43

1 Climatological records at Grand Junction, Colo.

2 Project lands only; Palisade, Mesa Company, and Orchard Mesa Irrigation Districts not included.

3 Project charges based on these deliveries; actual deliveries estimated as 5% greater.

4 Discharge of main canal at head less deliveries to Palisade, Mesa County, and Orchard Mesa Irrigation Districts.

5 Includes main canal loss on deliveries to Districts in 1919 and subsequent years, carried a distance of about 5 miles in canal which is largely lined.

6 Water supply, except on rare occasions, far in excess of project requirements.

7 Actual losses estimated as averaging 3% less than shown, due to under-measurement of deliveries.

8 Actual deliveries estimated as averaging 3% more than shown, due to under-measurement.

9 Final irrigable area—45 000 acres—will require material extension of lateral system.

TABLE 5.—Use of Water on the Huntley Irrigation Project of Montana.

(Average elevation, 3 000 ft.; practically all in earth.)

Calendar year.	Irrigated area, total for year.[8]	Area commanded by constructed canal system.[8]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[2] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Percentage of Total Diversions for Season.[3] Canal and lateral losses.[4]	Waste.[3]	Delivered to farms.[7]
1912	14 425	28 805	194	......	......	......	......	0.06	0.38	0.36	0.57	0.11	......	......	......	1.48	0.92	2.40	0.45	61	30	25	45	17	42	41
1913	15 798	28 805	194	......	......	......	......	0.09	0.26	0.64	0.44	0.10	......	......	......	1.53	0.62	2.15	0.47	63	33	34	33	14	42	44
1914	17 068	28 805	194	......	......	......	......	0.01	0.29	0.61	0.47	0.05	......	......	......	1.43	0.76	2.19	0.45	63	38	31	31	19	43	38
1915	18 203	30 813	212	......	......	......	0.03	0.16	0.01	0.26	0.40	0.11	......	......	......	0.97	1.18	2.15	0.21	60	33	32	35	26	40	34
1916	18 635	32 936	232	......	......	......	......	0.09	0.19	0.45	0.29	0.10	......	......	......	1.12	0.61	1.73	0.27	63	39	30	31	27	42	31
1917	19 122	31 892	232	......	......	......	......	0.00	0.08	0.60	0.36	0.07	......	......	......	1.11	0.65	1.76	0.44	63	42	36	22	33	34	33
1918	18 958	32 884	232	......	......	......	......	0.02	0.40	0.31	0.28	0.05	......	......	......	1.06	0.66	1.72	0.38	66	43	44	13	38[5]	32[6]	30
1919	19 310	32 885	232	......	......	......	......	0.39	0.53	0.44	0.27	0.02	......	......	......	1.65	0.56	2.21	......	64	46	45	9	42	24	34
1920	20 020	32 824	232	......	......	......	......	0.00	0.28	0.45	0.38	0.10	......	......	......	1.21	0.53	1.74	0.45	64	49	36	15	31	38	31
1921	18 800	31 632	232	......	......	......	......	0.00	0.32	0.59	0.40	0.11	......	......	......	1.42	0.57	1.99	0.22	64	39	41	20	28	38	34
1922	19 523	32 000	232	......	......	......	......	0.00	0.08	0.50	0.28	0.15	......	......	......	0.96	0.72	1.68	0.47	65	41	38	21	37	37	26
1923	20 000	32 000	232	......	......	......	......	0.02	0.26	0.44	0.19	0.10	......	......	......	1.01	1.06	2.07	0.25	65	42	24	34	50	22	28
1924	19 600	32 540	232	......	......	......	......	0.07	0.12	0.42	0.47	0.18	......	......	......	1.26	0.50	1.76	0.36	63	43	17	40	38	32	30
1925	18 939	32 540	232	......	......	......	......	0.28	0.13	0.53	0.43	0.13	......	......	......	1.50	0.44	1.94	0.39	65	34	30	36	45	20	35
1926	19 790	32 540	232	......	......	......	0.01	0.30	0.35	0.36	0.36	0.07	......	......	......	1.45	0.59	2.04	0.33	65	40	27	33	48	22	30

1 Climatological records at Huntley, 1912–18; at Ballantine, thereafter.

2 Project charges based on recorded deliveries; actual deliveries estimated as 15% greater.

3 Includes diversions for operation of pumps.

4 Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries.

5 Includes all waste except diversions for operation of pumps.

6 Diversions for operation of pumps only, no waste included.

7 Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

8 Final irrigable area estimated at about 32 000 acres, contains much adobe land which may prove infeasible of agricultural development.

TABLE 6.—USE OF WATER ON THE KING HILL IRRIGATION PROJECT OF IDAHO.

(Average elevation, 2 750 ft.; in volcanic ash and sandy soil; 40% concrete-lined or structures.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE. January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON. Canal and lateral losses.	Waste.[4]	Delivered to farms.
1920	4 780	11 340	...	...	...	...	...	...	...	...	...	...	...	...	[2]	4.69	0.19	4.88	0.29	60	77	10	7	6	...	...	...
1921	5 908	13 648	90	...	...	...	[3]	0.40	1.01	1.46	1.25	0.98	...	...	[3]	5.08	0.42	5.50	0.46	66	78	5	8	9	39	8	53
1922	6 440	13 648	90	...	...	...	0.02	0.89	1.24	1.33	1.34	0.75	...	...	...	5.57	0.31	5.88	0.52	65	74	6	11	9	39	2	59
1923	7 017	16 888	100	...	...	...	0.15	1.31	0.85	1.37	1.23	0.95	0.11	...	...	5.97	0.58	6.55	0.22	64	78	7	9	6	29	26	45
1924	6 235	16 888	100	...	...	...	0.58	1.66	1.56	1.67	1.63	1.12	0.39	0.04	...	8.65	0.14	8.79	0.15	64	77	5	12	6	31	18	51
1925	6 624	16 888	100	...	...	...	0.19	1.29	1.14	1.42	1.46	1.01	0.28	...	...	6.79	0.55	7.34	0.60	63	76	9	10	5	30	23	47
1926	6 364	16 890	93	...	...	...	0.68	1.63	1.75	1.67	1.73	1.09	0.38	...	...	8.84	0.18	9.02	0.23	64	75	9	11	5	[5]	[5]	58
1927	6 629	16 890	93	...	...	...	0.23	1.39	1.55	1.65	1.72	0.92	0.27	...	...	7.73	0.25	7.98	0.66	61	67	10	18	5	[5]	[5]	54

[1] Climatological records, Glenns Ferry, Idaho.

[2] Project operated prior to 1921 by King Hill Irrigation District, which again took over operation in January, 1927.

[3] Unrecorded diversions and deliveries, April 4 to May 18, 1921, prior to operation by U. S. Reclamation Bureau, not included.

[4] Project has a fixed uniform supply of available water, waste occurring largely at ends of season of waters that could not be conserved.

[5] Loss and waste not segregated in 1926 and 1927.

[6] Irrigable area, 16 890 acres, but will probably remain to a large degree unirrigated on account of unexpectedly heavy deliveries required.

TABLE 7.—USE OF WATER ON THE KLAMATH IRRIGATION PROJECT OF OREGON AND CALIFORNIA.

(Average elevation, 4 100 ft.; chiefly in sandy loam soil; 2 miles concrete-lined; Langell Valley areas not included.)

Calendar year.	Irrigated area, total for year.[2]	Area commanded by constructed canals.[13]	Miles of canals and laterals operated.[4]	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[5] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, May to September preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season.[6] Canal and lateral losses.[7, 9]	Waste.	Delivered to farms.[12]
1912	23 834	30 098	132	......	......	......	......	0.17	0.29	0.42	0.21	0.04	......	......	......	1.18	0.60	1.73	0.94	57	59	38	3	...	36	[8]	....
1913	18 928	28 900	146	......	......	......	......	0.26	0.43	0.23	0.21	0.04	......	......	......	1.17	0.40	1.57	0.71	60	66	32	2	...	......	[8]	....
1914	24 440	38 000	205	......	......	......	......	0.31	0.29	0.39	0.21	0.06	......	......	......	1.26	0.23	1.49	0.76	61	65	33	2	...	40	3	57
1915	27 254	38 000	205	......	......	......	0.01	0.15	0.47	0.30	0.16	0.03	......	......	......	1.12	0.19	1.31	0.55	60	58	40	2	...	50	6	44
1916	29 351	45 272	216	......	......	......	0.01	0.21	0.38	0.14	0.24	0.04	......	......	......	1.02	0.31	1.33	0.62	59	58	41	1	...	49	6	45
1917	33 635	44 715	216	......	......	......	......	......	0.38	0.31	0.26	0.03	......	......	......	0.98	0.17	1.15	0.65	61	60	39	1	...	42	8	50
1918	38 268	50 000	210	......	......	......	0.02	0.38	0.39	0.29	0.24	0.04	......	......	3	1.36	0.19	1.55	0.51	60	69	30	1	...	40	10	50
1919	42 881	50 000	216	......	......	......	......	0.36	0.34	0.32	0.28	0.02	......	......	3	1.32	0.16	1.48	0.62	61	74	25	1	...	47	6	47
1920	44 800	50 000	216	......	......	......	......	0.28	0.30	0.21	0.29	0.03	......	......	3	1.11	0.18	1.29	0.65	59	76	23	1	...	55	2	43
1921	43 883	50 000	216	......	......	......	......	0.15	0.36	0.27	0.28	0.05	......	......	3	1.11	0.15	1.26	0.97	60	79	20	1	...	52	2	46
1922	44 929	51 023	216	......	......	......	......	0.11	0.39	0.28	0.27	0.06	......	......	3	1.11	0.19	1.30	0.84	63	79	19	2	...	52	6	42
1923	46 624	54 171	277	......	......	......	......	0.31	0.30	0.22	0.31	0.07	......	......	3	1.21	0.23	1.44	0.77	59	82	16	2	...	24	30	46
1924	47 300	53 906	277	......	......	......	0.04	0.41	0.36	0.34	0.21	0.04	......	......	4	1.40	0.11	1.51	0.45	62	89	9	2	...	50[10]	9	41[10]
1925	47 400	55 167	277	......	......	......	......	0.26	0.33	0.35	0.23	0.02	......	......	4	1.19	0.40	1.59	0.84	59	82	15	3	...	48[10]	12	40[10]
1926	48 087	64 983	277	......	......	......	0.06	0.43	0.44	0.37	0.25	0.06	......	......	11	1.61	0.05	1.66	0.55	61	78	17	5	...	53[10]	6	41[10]

1 Climatological records, mean of Klamath Falls and Merrill, Ore.
2 Private districts included, 1918 and 1919, 5 000 acres; 1920, 6 700 acres; 1921, 7 800 acres; 1922, 1923, 1924, 8 929 acres; 1925, 8 498 acres; 1926, 9 883 acres.
3 Deliveries to private districts at heads of laterals instead of farms.
4 Private districts not included. Quantities shown are for project lands only.
5 Project charges based on these quantities; actual deliveries estimated as 20% greater.
6 Total diversions from Upper Klamath Lake, Lost River, and Lost River return flow.
7 Losses in private district laterals not included (about 16% of irrigated area in private districts 1918-23).
8 No record.
9 Actual losses estimated as averaging 8% less than shown, due to under-measurement of deliveries.
10 Based on an estimate of lateral loss on deliveries to private districts.
11 Based on quantities delivered to private district lands as well as project lands.
12 Actual deliveries estimated as averaging 8% more than shown, due to under-measurement.
13 Final irrigable area, 95 000 acres, including lands in private projects served by project main canals.

TABLE 8.—Use of Water on the Lower Yellowstone Irrigation Project of Montana.

(Average elevation, 1 900 ft.; all in earth.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[7]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[3] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April, preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Percentage of Total Diversions for Season. Canal and lateral losses.[5]	Waste.[4]	Delivered to farms.[6]
1912	5 068	37 880	126	.....	.....	.....	.....	0.00	0.08	0.98	0.11	0.01	0.01	.....	.....	1 19	1.05	2.24	0.56	58	12[2]	87[2]	1[2]	43	18	39
1913	7 660	36 578	133	.....	.....	.....	.....	0.00	0.71	0.46	0.09	0.06	0.02	.....	.....	1.34	0.64	1.98	0.28	63	23	75	2	52	14	34
1914	5 743	36 250	151	.....	.....	.....	.....	0.04	0.33	0.66	0.46	0.04	0.06	.....	.....	1.59	0.93	2.52	0.25	64	55	39	6	.....	.....	.....
1915	12 656	42 329	163	.....	.....	.....	.....	0.20	0.35	0.67	0.19	0.01	.....	.....	.....	1.42	1.09	2.51	0.23	59	42	56	2	43	12	45
1916	6 020	42 288	151	.....	.....	.....	.....	0.00	0.22	0.68	0.24	0.11	.....	.....	.....	1.25	1.22	2.47	0.44	62	58	40	2	48	24	28
1917	15 744	42 232	180	.....	.....	.....	.....	0.02	0.39	0.78	0.48	0.10	.....	.....	.....	1.77	0.35	2.12	0.43	63	50	42	8	43	11	46
1918	21 075	42 272	186	.....	.....	.....	.....	0.03	0.47	0.55	0.06	0.00	.....	.....	.....	1.11	0.72	1.83	0.36	64	40	53	7	47	8	45
1919	21 300	42 167	188	.....	.....	.....	.....	0.00	0.38	0.56	0.21	0.08	.....	.....	.....	1.23	0.45	1.68	0.36	66	52	43	5	57	5	38
1920	19 120	42 167	187	.....	.....	.....	.....	0.00	0.34	0.36	0.11	0.06	.....	.....	.....	0.87	0.63	1.50	0.47	65	43	49	8	50	15	35
1921	19 980	40 200	174	.....	.....	.....	.....	0.00	0.53	0.35	0.35	0.06	.....	.....	.....	1.29	0.53	1.82	0.37	64	51	35	14	44	16	40
1922	15 599	40 200	155	.....	.....	.....	.....	0.00	0.22	0.57	0.33	0.06	.....	.....	.....	1.18	0.83	2.01	0.14	66	48	31	21	40	23	37
1923	17 857	58 000	268	.....	.....	.....	.....	0.00	0.22	0 55	0 32	0.17	.....	.....	.....	1.26	0.96	2.22	0.28	66	42	27	31	41	34	25
1924	14 025	58 000	226	.....	.....	.....	.....	0.00	0.18	0.20	0.66	0 22	.....	.....	.....	1.26	0.71	1.97	0.32	62	41	4	55	41	36	23
1925	18 276	58 243	243	.....	.....	.....	.....	0.24	0.18	0.40	0 60	0.17	0.03	.....	.....	1.62	0.75	2.37	0.34	64	35	10	55	42	31	27
1926	23 331	58 248	255	.....	.....	.....	.....	0.42	0.24	0.59	0.36	0.18	0.01	.....	.....	1.80	0.64	2.44	0.18	64	33	32	35	37	29	34

[1] Climatological records at Savage, Mont.

[2] For entire cropped area of 17 500 acres.

[3] Project charges based on recorded deliveries; actual deliveries estimated as 5% greater.

[4] Large waste due to excess of supply over all project requirements and to silt-sluicing operations.

[5] Actual losses estimated as averaging 2% less than shown, due to under-measurement of deliveries.

[6] Actual deliveries estimated as averaging 2% more than shown, due to under-measurement.

[7] Final irrigable area, 59 349 acres.

TABLE 9.—USE OF WATER ON THE MILK RIVER IRRIGATION PROJECT OF MONTANA.

(Average elevation, 2 200 ft.; in earth, largely river bottom; Chinook Division not included.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[10]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[3] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March, preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Percentage of Total Diversions for Season.[4] Canal and lateral losses.[5,6]	Waste.	Delivered to farm.[9]
1912	352	7 800	30	....	....	....	....	....	....	0.62	0.11	0.05	0.05	....	....	0.83	1.38	2.21	0.15	....	10	89	1	....	....	....
1913	2 545	12 800	59	....	....	....	0.01	0.09	0.12	0.56	0.08	0.01	0.05	....	....	0.92	0.79	1.71	0.13	....	34	66	....	....	....	....
1914	2 201	13 440	53	....	....	....	0.14	0.40	0.06	0.20	....	....	....	....	....	0.80	1.15	1.95	0 11	57	44	55	1	41	6	58
1915	4 192[2]	22 200	86	....	....	....	0.12	0.26	0.06	0.17	0.01	0.01	0.05	....	....	0.70	1.13	1.83	0.20	58	60	39	1	61[7]	16	23
1916	5 518[2]	40 400	184	....	....	....	0.01	0.23	0.07	0.16	0.12	0.03	0 06	....	....	0.68	1.04	1.72	0.47	56	76	23	1	17	7	76
1917	11 058	45 000	204	....	....	....	....	0 11	0.34	0.40	0.09	0.01	0.03	0.03	....	1.01	0.44	1.45	0.24	57	65	33	2	25	10	65
1918	24 842	58 750	254	....	....	....	....	0.05	0.33	0.27	0.02	0.01	....	....	....	0.68	0.73	1.41	0.19	59	68	31	1	32	9	59
1919	25 485	58 900	317	....	....	....	....	0.08	0.40	0.27	0.06	0.02	0.01	....	....	0.84	0.59	1.43	0.18	59	80	19	1	31	8	61
1920	18 500	68 600	361	....	....	....	....	0.06	0.25	0.16	0.09	0.01	....	....	....	0.57	1.12	1.69	0.13	58	78	21	1	36	26	38
1921	11 440	69 200	247	....	....	....	0.01	0.05	0.29	0.08	0.08	0.01	0.02	....	....	0 54	1.14	1.68	0.14	58	75	23	2	33	14[8]	53
1922	12 011	66 300	255	....	....	....	0.01	0.03	0.15	0.23	0.05	0.01	0.03	....	....	0.51	0.94	1.45	0.25	59	81	16	3	26	10	64
1923	13 622	64 750	253	....	....	....	0.04	0.28	0.13	....	0.03	0.01	0.01	....	....	0.50	1.38	1.88	0.12	58	82	14	4	24	23	53
1924	14 594	65 300	284	....	....	....	0.03	0.15	0.11	0.21	0.08	0 02	0.01	....	....	0.61	1.11	1.72	0.14	56	76	14	10	39	42	19
1925	19 892	64 755	284	....	....	....	....	0.10	0.08	0.26	0.11	0.02	....	....	....	0.57	0.93	1.50	0.28	56	72	13	15	52	31	17
1926	16 485	73 252	287	....	....	....	0.03	0.19	0.15	0.25	0.07	0.03	....	....	....	0.72	0.72	1.44	0.16	58	54	36	10	63	17	20

[1] Mean of Glasgow and Malta, Mont., climatological records, 1912–26.
[2] From letter of March 17, 1925, from Project Superintendent.
[3] Project charges based on these deliveries; actual deliveries estimated as 10% greater.
[4] Total diversions from Milk River plus canal diversions from Nelson Reservoir.
[5] Includes loss in Main Canal on diversions for Nelson Reservoir, but not loss in Nelson Reservoir.
[6] Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries.
[7] Includes loss due to priming new canals and filling Nelson Reservoir below elevation of zero storage of 52.5 per cent.
[8] Includes waste to Bowdoin Lake and to Alkali Creek to facilitate river-bank protection work (6.9 per cent.).
[9] Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.
[10] Final irrigable area, 145 100 acres.

TABLE 10.—USE OF WATER ON THE SOUTH SIDE PUMPING DIVISION OF THE MINIDOKA IRRIGATION PROJECT, IDAHO.

(Average elevation, 4 200 ft.; in sandy loam soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[3]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.[2]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.	Delivered to farm.
1912	......	......	......	......	......	......	......	0.12	0.47	0.58	0.28	0.20	0.01	......	......	1.66	0.83	2.49	0.49	55	30	48	20	2	......	......	......
1913	32 084	47 770	250	......	......	......	......	0.24	0.34	0.58	0.53	0.31	0.01	......	......	2.01	0.61	2.62	0.22	58	37	41	20	2	......	......	......
1914	35 788	47 770	257	......	......	......	......	0.42	0.44	0.70	0.51	0.22	......	......	......	2.29	0.70	2.99	0.62	60	48	35	16	1	37	7	56
1915	38 188	49 000	274	......	......	......	0.22	0.22	0.52	0.74	0.59	0.41	0.04	......	......	2.74	0.50	3.24	0.22	59	53	32	14	1	36	4	60
1916	40 643	49 000	275	......	......	......	0.02	0.38	0.59	0.66	0.60	0.40	0.02	......	......	2.67	0.43	3.10	0.74	57	57	28	14	1	37	4	59
1917	43 220	49 000	276	......	......	......	......	0.08	0.63	0.75	0.67	0.30	0.01	......	......	2.44	0.46	2.90	0.45	58	57	29	14	...	33	6	61
1918	44 765	48 962	276	......	......	......	0.02	0.53	0.59	0.66	0.53	0.28	0.02	......	......	2.63	0.41	3.04	0.34	60	52	34	14	...	39	3	58
1919	45 000	48 962	276	......	......	......	......	0.48	0.63	0.52	0.42	0.28	0.02	......	......	2.35	0.39	2.74	0.26	60	51	29	20	...	41	1	58
1920	46 133	48 962	276	......	......	......	0.03	0.44	0.65	0.62	0.60	0.32	0.02	......	......	2.68	0.54	3.22	0.21	58	51	27	22	...	38	5	57
1921	46 573	48 962	276	......	......	......	......	0.19	0.40	0.67	0.62	0.20	0.05	......	......	2.13	0.77	2.90	0.38	58	44	29	27	...	38	6	56
1922	45 286	48 960	274	......	......	......	......	0.18	0.59	0.70	0.52	0.34	0.05	......	......	2.38	0.63	3.01	0.47	60	42	27	31	...	42	4	54
1923	45 166	48 960	274	......	......	......	......	0.35	0.42	0.65	0.60	0.46	0.01	......	......	2.49	1.06	3.55	0.20	58	44	26	30	...	41	3	56
1924	43 404	48 960	274	......	......	......	0.23	0.63	0.49	0.59	0.43	0.11	......	......	......	2.48	0.28	2.76	0.23	60	52	20	28	...	39	0	61
1925	45 037	48 960	274	......	......	......	0.19	0.46	0.55	0.71	0.65	0.25	......	......	......	2.81	0.61	3.42	0.36	59	50	29	21	...	41	3	56
1926	44 875	48 960	274	......	......	......	0.17	0.71	0.68	0.69	0.55	0.19	......	......	......	2.99	0.17	3.16	0.26	61	55	29	16	...	35	0	65

[1] Climatological records, 1912-17, Rupert; 1918-26, Burley, Idaho.

[2] Delivered by pumps at first lift. Does not include stock water in non-irrigation months, nor canal losses from Minidoka Dam to first lift, a distance of 14 miles.

[3] Final irrigable area, 48 960 acres.

TABLE 11.—USE OF WATER ON THE CARSON DIVISION OF THE NEWLANDS IRRIGATION PROJECT, NEVADA.

(Average elevation, 4 000 ft.; canals and laterals about 60% in sandy soil and 40% in clay.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[8]	Miles of canals and laterals operated	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[4]													Precipitation, March to October, in feet.[1]	Water delivered plus precipitation, March to October, in feet.	Precipitation, November to February, preceding, in feet.[1]	Mean temperature, March to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.[2]			Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay and pasture.	Small grains.	Furrow crops.	Canal and lateral losses.[5, 6]	Waste.	Delivered to farms.[7]
1912	34 970	....	240	......	......	0.13	0.31	0.55	0.66	0.16	0.18	0.19	0.09	......	[3]	2.22	0.31	2.53	0.03	57	80	17	3	41	1	58
1913	40 818	....	240	......	......	0.19	0.50	0.64	0.26	0.24	0.19	0.17	0.11	......	[3]	2.30	0.52	3.80	0.04	59	84	10	6	38	6	56
1914	36 466	....	245	......	......	0.07	0.50	0.65	0 53	0.63	0.44	0.22	0.12	......	......	3.16	0.09	3.25	0.34	60	89	8	3	27	22	51
1915	36 853	....	250	......	......	0.05	0.31	0.39	0.57	0.54	0.37	0.20	0.06	0.05	......	2.54	0.34	2.88	0.13	59	83	12	5	30	24	46
1916	35 710	....	250	......	......	0.04	0.41	0.62	0.63	0.66	0.36	0.20	0.01	0.01	......	2.94	0.20	3.14	0.30	58	86	12	2	32	23	45
1917	36 350	....	294	......	......	0.01	0.21	0.53	0.61	0.63	0.39	0.25	0.07	......	......	2.70	0.09	2.79	0.14	59	83	9	8	35	23	42
1918	37 718	....	294	......	......	0 05	0.32	0.60	0.52	0.52	0.33	0.22	0.03	......	......	2.59	0.42	3.01	0.09	60	82	16	2	37	18	45
1919	39 547	....	300	......	......	0.05	0.31	0.56	0.50	0.56	0.40	0.30	0.03	......	......	2.71	0.18	2.89	0.07	60	84	14	2	46	14	40
1920	40 667	....	310	......	......	......	0.16	0.55	0.49	0.62	0.39	0.31	0.07	......	......	2.59	0.32	2.91	0.12	58	88	10	2	45	14	41
1921	40 542	....	316	......	......	0.05	0.35	0.38	0 40	0.63	0.45	0.20	0.07	......	......	2.53	0.23	2.76	0.06	60	88	7	5	44	17	39
1922	39 300	....	338	......	......	......	0.09	0.53	0.53	9.65	0.44	0.36	0.09	0.02	......	2.71	0.21	2.92	0.30	59	88	7	5	43	19	38
1923	39 120	....	338	......	......	0.02	0.19	0.57	0.42	0.69	0.50	0.31	0.10	0.03	......	2.83	0.29	3.12	0.20	58	87	9	4	42	22	36
1924	38 310	....	338	......	......	0.02	0.41	0.67	0.62	0.75	0.58	0.30	0.03	0.02	......	3.40	0.13	3.53	0.15	59	85	11	4	39	4	57
1925	36 696	65 381	333	......	......	0.01	0.37	0.68	0.57	0.74	0.54	0.31	0.05	0.05	......	3.32	0.50	3.82	0.16	59	82	14	4	43	6	51
1926	39 834	65 277	333	......	......	0.06	0.43	0.75	0.64	0.63	0.58	0.24	0.04	0.03	......	3.40	0.16	3.56	0.17	60	81	16	3	39	4	57

1 Climatological records at Fallon, Nev.

2 Based on crop report for entire project. Grain percentage would be slightly higher for Carson Division alone.

3 Water shortage; Lahontan storage first available in 1914.

4 Project charges based on these deliveries; actual deliveries estimated as 10% greater.

5 Losses below Carson River diversion dam.

6 Actual losses estimated as averaging 5% less than shown, due to under-measurement of farm deliveries.

7 Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

8 Final irrigable area not determined, but may not exceed 70 000 acres.

TABLE 12.—USE OF WATER ON THE NORTH PLATTE IRRIGATION PROJECT OF NEBRASKA AND WYOMING.

(Average elevation, 4 100 ft.; chiefly in sandy loam soil; practically no lining.)

Calendar year.	Irrigated area, total for year.[9]	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[1] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, May to September, in feet.[2]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April preceding, in feet.[2]	Mean temperature, May to September, in degrees Fahrenheit.[2]	PERCENTAGE OF AREA IN DIFFERENT CROPS. Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[5] Canal and lateral losses.[6]	Waste.[7]	Delivered to farms.[8]
1912	50 250	70 000	602	......	......	......	......	0.15	0.75	0.72	0.23	0.40	......	......	......	2.25	1.23	3.48	0.53	61	47	33	20	39	1	60
1913	56 829	75 500	648	......	......	......	......	0.17	0.60	0.58	0.71	0.43	......	......	......	2.49	1.00	3.49	0.32	66	60	25	15	38	2	60
1914	60 532	75 500	651	......	......	......	......	0.31	0.56	0.78	0.80	0.45	0.02	......	......	2.92	0.38	3.30	0.56	66	62	17	21	45	1	54
1915	70 007	95 500	798	......	......	......	......	0.02	0.25	0.58	0.31	0.22	......	......	......	1.38	1.45	2.83	0.64	61	53	17	30	50	1	49
1916	75 620	96 000	800	......	......	......	......	0.29	0.40	0.56	0.60	0.32	......	......	......	2.17	0.65	2.82	0.24	64	51	24	25	43	1	56
1917	83 203	96 000	805	......	......	......	......	0.01	0.32	0.64	0.69	0.47	......	......	......	2.13	0.87	3.00	0.46	63	49	22	29	42	1	57
1918[3]	93 636	107 500	882	......	......	......	......	0.15	0.50	0.67	0.63	0.29	......	......	......	2.24	1.21	3.45	0.46	66	47	30	23	46	1	53
1919	95 248	107 500	882	......	......	......	......	0.23	0.61	0.65	0.57	0.22	......	......	......	2.28	0.62	2.90	0.42	67	42	29	29	46	6	48
1920	96 535	112 500	935	......	......	......	......	0.00	0.39	0.66	0.58	0.36	......	......	......	1.99	0.68	2.67	0.76	65	38	24	38	43	8	49
1921	99 199	112 500	942	......	......	......	......	0.16	0.21	0.67	0.67	0.39	......	......	......	2.10	0.75	2.85	0.37	67	36	27	37	45	5	50
1922	110 305	146 300	1 144	......	......	......	......	0.10	0.52	0.72	0.69	0.49	......	......	......	2.52	0.62	3.14	0.52	67	32	27	41	42	8	50
1923[4]	128 795	172 200	1 339	......	......	......	......	0.02	0.19	0.57	0.47	0.45	......	......	......	1.70	1.11	2.81	0.46	65	29	23	48	41	12	47
1924	130 121	206 200	1 451	......	......	......	......	0.19	0.52	0.80	0.90	0.44	......	......	......	2.85	0.53	3.38	0.51	64	29	23	48	41	15	44
1925	162 698	225 000	1 579	......	......	......	......	0.24	0.44	0.72	0.59	0.38	......	......	......	2.37	0.84	3.21	0.35	66	27	29	44	41	12	47
1926	170 831	237 000	1 579	......	......	......	......	0.10	0.36	0.59	0.70	0.37	......	......	......	2.12	0.90	3.02	0.42	65	30	29	41	......	......	......

[1] Does not include North Platte Canal and Colonization Company lands.

[2] Climatological records average of those at Mitchell (near) and Scottsbluff, Nebr.

[3] Fort Laramie unit included, 1918 and later years.

[4] Northport unit included, 1923 and later years.

[5] Total diversions from North Platte River at Whalen less waste at sand traps near diversion dam plus Northport Division diversions from Tri-State Canal plus outflow from Reservoirs 1, 2 and 3, minus inflow to these reservoirs. (Reservoirs 1 2 and 3 under main canal of Interstate Diversion).

[6] Does not include losses in Reservoirs 1, 2 and 3, losses above Northport diversion in Tri-State Canal, nor lateral losses in North Platte Canal and Colonization System (about 10% of total area), but does include losses on Lingle power diversions.

[7] Most of waste in upper reaches of main canals and utilized by other irrigators.

[8] Diversions to North Platte Canal and Colonization laterals considered as farm deliveries.

[9] Exclusive of North Platte Canal and Colonization Company lands, 16 000 acres irrigable, receiving water from Interstate Canal near its head; final irrigable area, 237 000 acres.

TABLE 13.—USE OF WATER ON THE OKANOGAN IRRIGATION PROJECT OF WASHINGTON.
(Average elevation, 1 000 ft.; 39 miles concrete-lined;* remainder in gravelly soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre. January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March, preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season. Canals and lateral losses.	Waste.	Delivered to farms.
1912	7 263	10 051	46	......	......	......	......	0.10	0.43	0.43	0.28	......	......	......	......	1.24	0.74	1.98	0.72	60	15	....	21	64	47	1	52
1913	7 700	10 084	44	......	......	......	......	0.20	0.30	0.49	0.56	0.01	......	......	......	1.56	0.53	2.09	0.31	62	18	....	8	74	38	0	62
1914	7 740	10 099	67†	......	......	......	......	0.55	0.62	0.64	0.74	0.04	......	......	......	2.59	0.41	3.00	0.66	62	21	....	3	76	28	1[4]	71
1915	7 800	10 099	72	......	......	......	0.02	0.30	0.55	0.61	0.76	0.10	0.02	0.02	......	2.38	0.50	2.88	0.62	62	22	....	8	70	24	1	75
1916	7 850	10 099	75	......	......	......	......	0.31	0.55	0.46	0.76	0.37	0.01	0.04	......	2.50	0.63	3.13	0.69	60	23	....	7	70	22[3]	2	76
1917	8 000	10 099	79	......	......	......	......	0.06	0.66	0.81	0.77	0.17	0.02	......	......	2.49	0.29	2.78	0.34	62	25	1	8	66	20	1	79
1918	6 402	10 099	69	......	......	......	......	0.23	0.32	0.16	0.10	0.15	0.03	......	[2]	0.99	0.42	1.41	0.49	63	23	2	5	70	34	2	64
1919	5 849	10 099	65	......	......	......	......	0.33	0.39	0.48	0.40	0.10	......	......	[2]	1.70	0.30	2.00	0.58	62	23	....	3	74	28	0	72
1920	5 436	8 599	66	......	......	......	......	0.05	0.21	0.33	0.23	0.11	0.04	......	[2]	0.97	0.44	1.41	0.24	62	15	....	3	82	38[5]	0	62
1921	5 644	8 599	67	......	......	......	......	0.39	0.68	0.73	0.74	0.40	......	0.01	......	2.95	0.29	3.24	0.76	62	14	....	3	83	24	0	76
1922	5 569	8 047	69	......	......	......	......	0.35	0.69	0.70	0.77	0.24	......	......	[2]	2.75	0.42	3.17	0.46	64	11	....	4	85	28	0	72
1923	5 162	7 654	69	......	......	......	0.01	0.44	0.16	0.73	0.73	0.57	......	......	[2]	2.64	0.70	3.34	0.31	63	8	....	4	88	33	1	66
1924	4 940	7 589	70	......	......	......	0.06	0.37	0.35	0.47	0.36	0.18	0.01	......	[2]	1.80	0.29	2.09	0.34	63	9	....	5	86	35[5]	0	65
1925	4 976	7 589	70	......	......	......	......	0.28	0.41	0.57	0.56	0.39	......	......	......	2.21	0.28	2.49	0.46	64	4	0	3	93	30	0	70
1926	4 581	7 300	70	......	......	......	0.06	0.33	0.42	0.52	0.43	0.25	......	......	[2]	2.01	0.37	2.38	0.49	64	2	....	3	95	24	0	76

* Six miles lined in 1912 and increased gradually to 39 miles in 1925.

† Distribution system extended to provide individual farm deliveries. Shrinkage in irrigable and irrigated areas resulted from inadequate water supply which became evident in the 1918–26 period of sub-normal stream flow.

1 Climatological record at Omak, Wash.

2 Average for all lands under project; shortages for the new lands occurred in 1918–20, 1922–24, and 1926.

3 Includes some water used in October and November for concrete lining and unrecorded deliveries.

4 Some project waste combined with reservoir spill in records.

5 Percentage of losses increased by shortage of water.

TABLE 14.—USE OF WATER ON THE ORLAND IRRIGATION PROJECT OF CALIFORNIA.

(Average elevation, 250 ft.; 89 miles concrete-lined; remainder in clay and loam soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[7]	Miles of canals and laterals operated.[3]	Delivered to Farms, in Hundredths of Acre-Feet per Acre.													Precipitation, March to October, inclusive, in feet.[1]	Water delivered plus precipitation, March to October, inclusive, in feet.	Precipitation, November to February preceding, in feet.[1]	Mean temperature, March to October, inclusive, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.	Delivered to farms.
1912	4 230	14 200	81	......	......	0.05	0.28	0.68	0.81	0.78	0.76	0.29	0.30	......	......	3.95	0.93	4.88	0.30	67	82	....	8	10	20	31	49
1913	6 616	14 300	84	......	......	0.31	0.36	0.67	0.62	0.70	0.34	......	......	......	......	3.00	0.35	3.35	0.47	71	85	....	5	10	....	....	....
1914	7 354	14 300	86	......	......	0.03	0.22	0.73	0.74	0.80	0.78	0.60	0.18	......	......	4.08	0.26	4.34	2.13	69	82	....	6	12	....	....	....
1915	8 928	20 320	108	......	......	......	0.01	0.21	0.80	0.76	0.69	0.67	0.26	......	......	3.40	0.52	3.92	1.75	71	74	....	10	16	27	15	58
1916	9 357	20 533	121	......	......	......	0.53	0.79	0.80	0.72	0.70	0.51	0.02	......	......	4.07	0.35	4.42	1.27	69	65	....	16	19	20	16	64
1917	12 729	20 480	128	......	0.06	0.10	0.37	0.56	0.56	0.70	0.64	0.47	0.02	0.01	......	3.49	0.12	3.61	0.76	71	54	....	30	16	27	13	60
1918	14 764	20 480	128	......	......	......	0.18	0.48	0.37	0.39	0.24	0.23	0.08	......	[2]	1.91	0.61	2.52	0.60	69	50	9	25	16	33	5	62
1919	15 203	20 480	128	......	......	......	0.33	0.58	0.56	0.59	0.53	0.35	0.02	......	......	2.96	0.24	3.20	0.90	69	48	11	22	19	32	5	63
1920	13 872	20 533	128	0.01	0.06	0.12	0.35	0.34	0.26	0.20	0.11	0.04	......	......	[2]	1.49	0.44	1.93	0.28	68	56	9	13	22	37[4]	2	61
1921	14 697	20 533	139	......	......	......	0.34	0.50	0.51	0.57	0.56	0.43	0.09	0.01	......	3.01	0.37	3.38	1.41	68	53	7	17	23	26	9	65
1922	15 119	20 665	139	.....	......	......	0.47	0.54	0.57	0.64	0.59	0.49	0.04	......	......	3.34	0.26	3.60	0.99	69	56	5	14	25	25	9	66
1923	15 500	20 665	139	......	......	0.24	0.17	0.64	0.52	0.56	0.54	0.36	0.01	0.02	......	3.06	0.42	3.48	0.90	69	51	5	15	29	25	10	65
1924	11 962	20 659	114	......	0.02	0.10	0.25	0.26	0.12	0.01	0.01	0.01	0.01	0.06	0.03[2]	0.88	0.46	1.34	0.47	70	62	3	4	31	26[5]	13[6]	61
1925	13 955	20 659	137	0.09	0.05	0.20	0.08	0.39	0.60	0.77	0.65	0.43	0.09	0.04	......	3.39	0.71	4.10	1.22	69	50	4	16	30	27	11	62
1926	14 674	20 754	133	0.01	......	0.22	0.14	0.57	0.65	0.60	0.46	0.29	0.03	0.02	......	2.98	0.50	3.48	0.96	72	58	4	18	20	25	5	70

[1] Climatological records at Orland, Calif.

[2] Severe water shortages, supply in some other years slightly limited.

[3] Covers distribution system only, Seven Mile Reservoir feeder canal omitted.

[4] High rate of loss due to water shortage and resulting small canal discharges.

[5] Severe water shortage but rate of loss normal as most of canal system was dry from July 8 to November 9.

[6] Practically all waste occurred in November and December, after water shortage was over.

[7] Final irrigable area, 20 659 acres; irrigated area in past limited by water supply; additional storage to become available in 1929.

TABLE 15.—USE OF WATER ON THE RIO GRANDE IRRIGATION PROJECT OF NEW MEXICO AND TEXAS.
(Average elevation, 3 700 ft.; 10 miles concrete-lined, remainder chiefly in silt.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[11]	Miles of canals and laterals operated.[9]	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[2 4] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, February to November, in feet	Water delivered plus precipitation, February to November, in feet.	Precipitation, December to January, preceding, in feet.[1]	Mean temperature, February to November, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season. Canal and lateral losses.[5 8]	Waste.[6]	Delivered to farms.[10]
1912	23 115	25 000	6	......	......	....	......	....	......	......	......	......	......	......	......	....	0.77	......	0.03	64	72	6	20	2	....	....	....
1913	27 723	35 000	37	......	0.19	0.83	0.86	1.07	0.93	0.22	0.05	0.04	0.16	......	......	4.35	0.67	5.02	0.08	64	71	7	18	4	7[5]	10	83
1914	28 442	40 000	37	0.17	0.39	0.52	0.82	0.96	0.80	0.47	0.80	0.47	0.28	......	......	5.68	0.88	6.56	0.08	65	64	8	22	5	26[7]	7	74
1915	33 876	45 000	37	......	0.05	0.15	0.91	0.99	1.18	1.02	0.77	0.56	0.17	......	0.10	5.90	0.64	6.54	0.39	64	66	9	20	5	5	17	78
1916	62 513	85 000	63	......	0.47	0.77	0.65	0.98	1.11	1.05	0.67	0.70	0.28	......	0.05	6.73	0.59	7.32	0.06	66	63	14	22	2	6	7	87
1917	64 308	88 000	86	0.01	0.37	0.72	0.85	0.93	1.18	1.13	0.77	1.05	0.68	0.29	0.40	8.38	0.48	8.86	0.04	65	42	22	34	2	....	....	....
1918	64 781	92 300	153	......	0.11	0.65	0.84	0.97	0.98	0.95	0.75	0.70	0.09	0.11	......	6.15	0.51	6.66	0.07	65	47	19	32	2	15[5]	20	65
1919	70 012	102 850	310	......	......	0.11	0.30	0.44	0.50	0.40	0.44	0.22	0.03	0.06	[3]	2.50	0.72	3.22	0.08	65	48	18	32	2	30	41	29
1920	82 956	115 000	397	0.03	0.03	0.22	0.45	0.57	0.43	0.44	0.48	0.33	0.13	0.03	0.05	3.09	0.53	3.62	0.09	65	43	14	41	2	30	40	30
1921	77 151	115 000	416	......	0.02	0.23	0.37	0.42	0.44	0.37	0.29	0.23	0.11	0.06	0.01	2.55	0.59	3.14	0.00	66	48	15	35	2	35	40	25
1922	74 325	116 000	416	......	0.06	0.13	0.31	0.30	0.43	0.45	0.45	0.35	0.15	0.04	0.04	2.71	0.36	3.07	0.05	66	47	11	40	2	41	38	21
1923	86 583	120 000	537	......	0.02	0.08	0.32	0.41	0.41	0.35	0.29	0.19	0.08	0.02	0.01	2.18	0.66	2.84	0.05	65	38	3	57	2	30	51	19
1924	110 318	142 000	584	......	0.02	0.09	0.34	0.38	0.38	0.23	0.38	0.23	0.07	0.02	0.02	2.18	0.48	2.66	0.10	66	31	1	66	2	38	44	18
1925	130 911	150 000	607	......	0.05	0.18	0.38	0.34	0.34	0.37	0.23	0.20	0.03	0.02	0.02	2.16	0.56	2.72	0.04	66	22	....	77	1	43	34	23
1926	142 523	150 000	612	......	0.07	0.13	0.27	0.26	0.33	0.24	0.35	0.26	0.01	0.01	0.01	1.91	0.93	2.84	0.07	66	21	1	77	1	47	28	25

1 Mean of climatological records at El Paso, Tex., and Agricultural Coll., New Mexico.

2 Includes losses in community ditches, 1912 to 1918, by considering deliveries to community ditches as deliveries to farms. No data on use under community canals.

3 Charges for water deliveries changed from flat rate to acre-feet basis which was thereafter used.

4 Project charges based on these deliveries; actual deliveries estimated as 25% greater.

5 Loss in community ditches not included, 1912 to 1918.

6 Returned to river and largely available for re-diversion.

7 Waste unmeasured, included in loss.

8 Actual losses estimated as averaging 7% less than shown, due to under-measurement of deliveries.

9 Mileage increase disproportionate to irrigated area, due to gradual taking over of community ditches.

10 Actual deliveries estimated as averaging 7% more than shown, due to under-measurement.

11 Final irrigable area, 155 000 acres.

TABLE 16.—USE OF WATER ON THE FRANNIE DIVISION OF THE SHOSHONE IRRIGATION PROJECT, WYOMING.

(Average elevation 4 150 ft.; in sand and clay loam, no lining.)

Calender year.	Irrigated area, total for year.	Area commanded by constructed canals.[4]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[1] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to September, in feet.[5]	Water delivered plus precipitation, April to September, in feet.	Precipitation, October to March, preceding, in feet.	Mean temperature, April to September, in degrees Fahrenheit.	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season. Canal and lateral losses.[2]	Waste.	Delivered to farms.[3]
1917	2 300	5 000	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.53	0.29	0.82	0.10	58	....	95	5	....	....	....	....
1918	4 729	13 007	.....	.....	.....	.....	.....	0.08	0.86	1.00	0.72	0.19	.....	.....	.....	2.85	0.50	3.35	0.08	58	6	91	3	....	....	....	....
1919	6 943	17 500	.....	.....	.....	.....	0.04	0.45	0.82	0.98	0.53	0.23	.....	.....	.....	3.05	0.11	3.16	0.12	62	21	70	9	....	....	....	....
1920	10 460	22 484	.....	.....	.....	.....	.....	0.16	0.66	0.82	0.66	0.29	0.04	.....	.....	2.63	0.16	2.79	0.22	59	42	47	9	....	....	....	....
1921	10 844	27 473	.....	.....	.....	.....	0.11	0.44	0.54	0.73	0.41	0.23	0.17	.....	.....	2.63	0.28	2 91	0.10	61	63	29	8	....	....	....	....
1922	10 152	27 410	175	.....	.....	.....	.....	0.21	0.73	0.71	0.53	0.29	0.14	.....	.....	2.61	0.36	2.97	0.08	61	72	20	8	....	33	17	50
1923	8 197	27 057	170	.....	.....	.....	.....	0.28	0.78	0.82	0.52	0.27	0.03	.....	.....	2.70	0.46	3.16	0 11	60	75	16	9	....	38	17	45
1924	6 750	20 000	171	.....	.....	.....	.....	0.34	0.41	0.47	0.47	0.19	0.01	.....	.....	1.89	0.17	2.06	0.14	58	72	15	13	....	46	24	30
1925	7 054	20 063	171	.....	.....	.....	0.10	0.39	0.33	0.64	0.42	0 14	.....	.....	.....	2.02	0.44	2.46	0.07	61	67	22	11	....	45	28	27
1226	7 663	20 063	143	.....	.....	.....	0.01	0.21	0.53	0.52	0.39	0.06	.....	.....	.....	1.72	0.52	2.24	0.13	59	71	14	15	....	49	17	34

[1] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[2] Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries. Available data prior to 1922 do not segregate losses for Frannie Division. Indicated data after 1921 are based on losses within division only.

[3] Actual deliveries estimated as averaging 5% more than here shown, due to under-measurement.

[4] Final irrigable area, 28 000 acres; indicated reduction in past commanded areas due to elimination of poor land.

[5] Climatological records at Deaver, Wyo.

TABLE 17.—USE OF WATER ON THE GARLAND DIVISION OF THE SHOSHONE IRRIGATION PROJECT, WYOMING.

(Average elevation, 4 400 ft.; in gravel and loam; 4 miles of tunnel and other concrete-lined conduit.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[1]													Precipitation, April to September, in feet.[7]	Water delivered plus precipitation, April to September, in feet.	Precipitation, October to March preceding, in feet.	Mean temperature, April to September, in degrees Fahrenheit.	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[2, 3]	Waste.	Delivered to farms.[5]
1912	16 524	41 331	242	......	......	......	0.00	0.22	0.58	0.48	0.27	0.10	0.01	......	......	1.66	0.49	2.15	0.07	56	60	37	3	...	36	8	56
1913	19 423	41 172	245	......	......	......	0.01	0.40	0.41	0.52	0.48	0.19	0.07	......	......	2.08	0.32	2.40	0.12	60	66	31	3	....	36	5	59
1914	22 226	41 166	245	......	......	......	0.04	0.35	0.45	0.72	0.48	0.15	0.19	......	......	2.38	0.46	2.84	0.06	59	67	30	3	....	37	5	58
1915	25 753	42 816	247	......	......	......	0.19	0.31	0.34	0.61	0.31	0.23	0.01	0 12	......	2.12	0.71	2.83	0.05	57	58	35	7	....	35	8	57
1916	29 977	42 623	266	......	......	......	0.07	0.32	0.46	0.75	0.45	0.23	0.01	0.05	......	2.34	0.25	2.59	0.07	58	54	40	6	....	35	9	56
1917	32 764	43 263	271	......	......	......	0.00	0.08	0.52	0.80	0.48	0.15	0.01	0.06	......	2.10	0.45	2.55	0.16	57	52	41	7	....	28	5	67
1918	33 552	42 299	280	......	......	......	......	0.27	0.68	0.55	0.44	0.17	......	......	......	2.11	0.60	2.71	0.09	57	52	43	5	....	37	10	53
1919	34 697	43 400	280	......	......	.....	0.08	0.58	0 61	0.81	0.42	0.27	......	......	......	2.77	0.07	2.84	0.07	61	52	41	7	....	34	10	56
1920	35 175	43 284	280	......	......	......	......	0.29	0.69	0.66	0.53	0.32	......	......	......	2.49	0.14	2.63	0.07	57	61	30	9	....	36	6	58
1921	34 571	43 649	280	......	......	......	0.30	0.11	0.53	0.69	0.45	0.20	0.14	......	......	2.43	0.21	2.64	0.02	60	63	26	11	....	44	6	50
1922	32 717	43 813	282	......	......	......	......	0.23	0.59	0.67	0.41	0.21	0.13	......	......	2.24	0.46	2.70	0.08	60	58	26	16	....	45	5	50
1923	30 468	43 288	282	......	......	......	......	0.36	0.54	0.72	0.44	0.20	......	......	......	2.26	0.35	2.61	0.13	59	61	23	16	....	41	7	52
1924	29 757	42 000	279	......	......	......	......	0.49	0.52	0.63	0.68	0.24	0.01	......	......	2.57	0.18	2 75	0.14	57	66	16	18	....	41	5	54
1925	29 649	41 923	277	......	......	......	0.14	0.50	0.51	0.80	0.54	0.26	0.01	......	......	2.76	0.34	3.10	0.08	60	66	18	16	....	39	7	54
1926	30 457	41 923	277	......	......	......	......	0.29	0.53	0.63	0.50	0.13	0.01	......	......	2.09	0.55	2.64	0.09	58	60	13	27	....	[4]	[4]	....

1 Project charges based on these deliveries; actual deliveries estimated as 10% greater.

2 Includes losses from Ralston Regulating Reservoir. From 1918-21, inclusive, represents combined loss on Garland and Frannie Divisions. After 1921, represents losses within Garland Division alone, with aggregate quantity of water delivered to Frannie Division deducted from diversion.

3 Actual losses estimated as averaging 6% less than shown, due to under-measurement of deliveries.

4 Losses and waste not segregated, the total amounting to 46 per cent.

5 Actual deliveries estimated as averaging 6% more than shown, due to under-measurement.

6 Final irrigable area, 42 000 acres; indicated past reductions in commanded areas, due to elimination of poor land.

7 Climatological records at Powell, Wyo.

TABLE 18.—USE OF WATER ON THE FORT SHAW DIVISION OF THE SUN RIVER IRRIGATION PROJECT, MONTANA.

(Average elevation, 3 700 ft.; wholly in earth.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canal system.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[2]													Precipitation, May to October, in feet.[1]	Water delivered plus precipitation, May to October, in feet.	Precipitation, November to April preceding, in feet.[1]	Mean temperature, May to October, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[3]	Waste.[4]	Delivered to farms.[5]
1912	6 824	16 346	120	......	......	......	......	......	0.80	0.72	0.18	0.01	......	......	......	1.71	0.73	2.44	0.14	......	23	73	4	....	20	27	53
1913	7 419	16 346	120	......	......	......	......	0.02	0.52	0.57	0.22	0.18	......	......	......	1.51	0.82	2.33	0.20	58	38	59	3	....	38	16	46
1914	6 613	16 346	110	......	......	......	......	0.03	0.29	0.92	0.36	0.12	0.01	......	......	1.73	0.71	2.44	0.22	59	51	46	3	....	40	14	46
1915	4 261	16 346	100	......	......	......	......	0.54	0.17	0.05	0.24	0.09	......	......	......	1.09	1.16	2.25	0.24	56	63	33	4	....	44	28	28
1916	4 717	16 332	100	......	......	......	......	0.14	0.26	0.14	0.58	0.09	0.01	......	......	1.22	1.22	2.44	0.15	56	71	26	3	....	42	26	32
1917	6 675	16 224	100	......	......	......	......	0.01	0.15	0.78	0.41	0.01	......	......	......	1.36	0.61	1.97	0.21	58	73	24	3	....	38	27	35
1918	7 569	16 095	96	......	......	......	......	0.21	0.68	0.32	0.26	0.01	......	......	......	1.48	0.42	1.90	0.15	59	63	35	2	....	40	23	37
1919	8 186	14 532	98	......	......	......	0.03	0.50	0.58	0.62	0.27	0.08	......	......	......	2.08	0.48	2.56	0.16	58	75	23	2	....	34	15	51
1920	8 048	13 935	100	......	......	......	......	......	0.54	0.29	0.40	0.01	......	......	......	1.25	0.50	1.75	0.36	58	75	22	3	....	35	24	41
1921	8 912	13 785	100	......	......	......	......	0.02	0.55	0.53	0.30	0.04	0.02	......	......	1.46	0.50	1.96	0.14	59	76	20	4	....	36	23	41
1922	8 115	13 701	100	......	......	......	......	......	0.41	0.53	0.40	0.06	0.02	......	......	1.42	0.61	2.03	0.40	59	82	12	6	....	29	31	40
1923	6 470	13 912	95	......	......	......	......	0.05	0.56	0.31	0.43	0.08	0.03	......	......	1.46	1.10	2.56	0.23	56	79	16	5	....	37	28	35
1924	7 888	13 902	101	......	......	......	......	0.31	0.34	0.55	0.44	0.06	0.01	......	......	1.71	0.46	2.17	0.17	57	80	10	10	....	39	38	23
1925	7 456	13 902	101	......	......	......	......	0.30	0.23	0.66	0.36	0.05	......	......	......	1.60	0.69	2.29	0.34	56	76	15	9	....	37	29	34
1926	7 183	13 902	101	......	......	......	......	0.17	0.45	0.62	0.29	0.04	......	......	......	1.57	0.63	2.20	0.11	59	71	21	8	....	35	18	47

[1] Climatological records at Fort Shaw, Mont.

[2] Project charges based on these deliveries; actual deliveries estimated as 5% greater.

[3] Actual losses estimated as averaging 2% less than shown, due to under-measurement of deliveries.

[4] Waste largely rediverted by prior rights on Sun River below Fort Shaw Division.

[5] Actual deliveries estimated as averaging 2% more than shown, due to under-measurement.

[6] Final irrigable area about 14 000 acres.

TABLE 19.—USE OF WATER ON THE GREENFIELDS DIVISION OF THE SUN RIVER IRRIGATION PROJECT, MONTANA.

(Average elevation, 3 700 ft.; lined in a few isolated stretches, remainder located in gravel and clay soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canal system.[7]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[1 2]													Precipitation, May to October, in feet.	Water delivered plus precipitation, May to October, in feet.	Precipitation, November to April, preceding, in feet.	Mean temperature, May to October, in degrees Fahrenheit.[8]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.[3]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[4 5]	Waste.	Delivered to farms.[6]
1919	3 311	24 300	148	......	......	......	......	0.00	0.72	0.72	0.00	0.00	0.00	......	......	1.44	0.48	1.92	0.16	58	48	50	2	...	46	1	58
1920	6 729	24 300	150	......	......	......	......	0.00	0.33	0.87	0.14	0.02	0.00	......	......	1.36	0.45	1.81	0.36	58	14	85	1	...	34	21	45
1921	11 849	25 150	167	...	......	...	......	0.01	0.73	0.53	0.05	0.01	0.01	......	......	1.34	0.49	1.83	0.17	57	13	86	1	...	27	13	60
1922	12 422	27 473	167	......	......	......	......	0.00	0.21	0.82	0.06	0.01	0.02	......	......	1.12	0.69	1.81	0.50	59	13	86	1	...	11	26	63
1923	2 624	28 304	164	......	......	......	......	0.00	0.00	0.58	0.60	0.06	0.40	......	......	1.64	0.99	2.63	0.36	57	32	66	2	...	27	50	23
1924	13 741	42 920	236	......	......	......	......	0.21	0.29	0.69	0.14	0.01	0.02	......	......	1.34	0.51	1.85	0.27	56	20	77	3	...	32	23	45
1925	13 012	42 920	241	......	......	......	......	0.23	0.18	0.74	0.05	......	......	......	......	1.20	0.78	1.98	0.33	55	21	78	1	...	36	20	44
1926	15 958	41 975	251	......	......	......	......	0.16	0.38	0.28	0.01	0.00	......	......	......	0.83	0.78	1.61	0.12	56	25	73	2	...	34	21	45

[1] Data cover Part No. 1 of Greenfields Division only; in general, about 94% of indicated total irrigated area.
[2] Project charges based on these deliveries; actual deliveries estimated as 5% greater.
[3] Measured at Fairfield Drop, losses from Sun River Diversion Dam to Drop excessive on account of small part of division area irrigated.
[4] Below Fairfield Drop, only.
[5] Actual losses estimated as averaging 2% less than shown, due to under-measurement of deliveries.
[6] Actual deliveries estimated as averaging 2% more than shown, due to under-measurement.
[7] Final irrigable area, 100 000 acres.
[8] Climatological records at Fort Shaw, Mont. for 1919; other years at Fairfield, Mont.

TABLE 20.—USE OF WATER ON THE UMATILLA IRRIGATION PROJECT OF OREGON.

(Average elevation, 470 ft.; 157 miles of pipe or lined canal, remainder in sandy loam.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[2] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, March to October, in feet.[1]	Water delivered plus precipitation, March to October, in feet.	Precipitation, November to February, preceding, in feet.[1]	Mean temperature, March to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season.[4] Canal and lateral losses.[5,7]	Waste.	Delivered to farms.[8]
1912	4 600	17 258	100	......	......	......	0.96	1.52	1.65	1.78	1.62	0.72	......	......	......	8.25	0.43	8.68	0.31	59	64	....	13	23	28	2[6]	70
1913	5 006	16 652	112	......	......	0.02	1.09	1.59	1.71	1.31	1.63	1.10	......	......	......	8.45	0.38	8.83	0.23	60	47	14	8	31	27	2	71
1914	5 100	17 000	120	......	......	0.21	1.09	1.38	1.42	1.29	1.20	0.52	......	......	......	7.11	0.27	7.38	0.39	62	60	4	8	28	40	2	58
1915	5 306	17 000	127	......	......	0.05	0.73	0.66	1.26	1.17	1.15	0.53	0.02	......	......	5.57	0.30	5.87	0.19	63	66	2	7	25	44	28	28
1916	5 477	17 000	139	......	......	......	0.44	0.99	1.15	1.05	1.26	0.82	0.05	......	......	5.76	0.53	6.29	0.58	59	75	....	7	18	36	8	56
1917	7 327	26 300	164	......	......	......	0.25	0.88	1.34	1.50	1.27	0.64	0.26	0.05	......	6.19	0.31	6.50	0.38	59	78	....	7	15	25[5]	23	52
1918	9 100	24 732	167	......	......	......	0.77	1.02	0.98	0.94	0.82	0.65	0.11	......	......	5.29	0.28	5.57	0.40	61	82	1	6	11	32	16	52
1919	10 533	24 625	167	......	......	0.05	0.62	1.08	0.99	0.98	0.95	0.44	0.13	......	......	5.24	0.24	5.48	0.36	60	86	1	5	8	33	23	44
1920	12 028	24 464	168	......	......	......	0.18	1.05	0.77	1.09	0.87	0.21	0.04	......	......	4.21	0.53	4.74	0.39	60	89	....	4	7	31	25	44
1921	13 145	24 400	174	......	......	......	0.60	0.89	0.86	0.91	0.84	0.26	0.01	......	......	4.37	0.25	4.62	0.48	60	89	1	4	6	31	24	45
1922	13 273	24 592	180	......	......	......	0.21	0.92	0.93	1.09	0.83	0.47	0.02	......	......	4.47	0.30	4.77	0.44	61	88	1	6	5	34	18	48
1923	13 330	24 587	188	......	......	......	0.51	1.02	0.78	0.94	0.95	0.44	......	......	......	4.64	0.54	5.18	0.33	61	89	1	6	4	32	19	49
1924	13 134	24 587	193	......	......	......	0.75	0.92	0.69	1.11	0.91	0.14	......	......	......	4.52	0.28	4.80	0.25	61	89	1	6	4	36	12	52
1925	13 345	24 587	192	......	......	0.11	0.84	1.07	1.10	1.13	0.93	0.35	......	......	......	5.53	0.23	5.76	0.36	62	90	2	5	3	28	12	60
1926	12 549	24 587	192	......	......	0.12	1.02	0.98	1.08	1.00	0.77	0.29	......	......	......	5.26	0.33	5.59	0.35	62	91	3	5	1	28	8	64
1927	11 462	24 587	......	......	......	......	0.50	1.05	1.16	1.38	1.22	0.29	......	......	......	5.70	0.50	6.20	0.53	60	85	5	6	4	24	16	60

1 Climatological record at Hermiston, Ore.

2 Project charges based on these deliveries; actual deliveries estimated as 10% greater.

3 Not including feed canal, which is 25 miles long with an average annual loss of 9 per cent.

4 Sum of discharges at heads of Maxwell, A, and West Division Main Canals.

5 Loss in West Division Canals estimated at 2%; system is largely concrete-lined or piped.

6 Includes some water used for construction purposes.

7 Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries.

8 Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

9 Final irrigable area, 28 300 acres.

TABLE 21.—USE OF WATER ON THE UNCOMPAHGRE IRRIGATION PROJECT OF COLORADO.

(... lined tunnel and canal; about one-half of canals in adobe and shale,

TABLE 21.—USE OF WATER ON THE UNCOMPAHGRE IRRIGATION PROJECT OF COLORADO.

(Average elevation, 5 500 ft.; 11 miles concrete-lined tunnel, and canal; about one-half of canals in adobe and shale, remainder in sand and gravel.)

Calendar year.	Irrigated area, total for year.[3]	Area commanded by constructed canals.[11]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[4 5] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season.[6] Canal and lateral losses.[5 7 8]	Waste.	Delivered to farms.[10]
1912	27 887	44 500	211	......	......	......	[2]	0.93	1.02	0.83	0.92	0.72	0.39	......	......	4.81	0.64	5.45	0.33	56	43	22	23	12	5[9]	....	95
1913	31 428	48 000	228	......	......	......	[2]	0.89	1.04	0.98	0.98	0.75	0.45	......	......	5.09	0.33	5.42	0.16	61	41	24	26	9	5[9]	....	95
1914	33 873	52 338	280	......	......	......	[2]	0.81	1.07	1.05	0.91	0.71	0.51	......	......	5.06	0.85	5.91	0.38	60	39	33	20	8	8[9]	....	92
1915	41 463	62 147	356	......	......	......	0.35	0.97	1.13	1.11	1.04	0.58	0.41	......	......	5.58	0.47	6.05	0.19	60	44	31	18	7	12[9]	....	88
1916	49 275	77 713	406	......	......	......	0.43	1.17	1.27	1.26	0.85	0.77	0.33	......	......	6.08	0.81	6.89	0.32	60	44	32	17	7	2	7	91
1917	53 108	90 000	415	......	......	......	0.23	0.72	1.34	1.35	1.11	0.77	0.44	......	......	5.96	0.53	6.49	0.24	59	42	31	22	5	3	11	86
1918	58 270	100 000	413	......	......	......	0.39	1.32	1.31	1.20	0.95	0.68	0.45	......	......	6.30	0.55	6.85	0.22	60	43	32	21	4	7	6	87
1919	60 906	100 000	417	......	......	......	0.41	1.30	1.21	1.29	1.01	0.71	0.49	......	......	6.42	0.46	6.88	0.34	61	48	29	19	4	6	9	85
1920	64 186	100 000	448	......	......	......	0.14	0.88	1.27	1.17	1.06	0.86	0.32	......	......	5.70	0.68	6.38	0.26	59	52	24	21	3	5	12	83
1921	63 759	97 410	455	......	......	......	0.52	1.32	1.10	1.34	0.95	0.76	0.53	......	......	6.52	0.65	7.17	0.22	62	49	26	22	3	9	12	79
1922	64 730	97 410	468	......	......	......	0.45	1.21	1.33	1.12	1.08	0.84	0.50	......	......	6.53	0.29	6.82	0.26	61	47	21	29	3	5	9	86
1923	64 324	97 064	488	......	......	......	0.45	0.93	1.16	1.14	0.74	0.56	0.13	......	......	5.11	0.55	5.66	0.25	60	45	24	28	3	21	12	67
1924	62 184	95 202	532	......	......	......	0.39	1.11	1.00	1.10	0.76	0.45	0.08	......	......	4.89	0.52	5.41	0.26	60	46	18	33	3	20	9	71
1925	61 637	95 202	532	......	......	......	0.58	1.16	0.98	1.11	0.72	0.18	0.11	......	......	4.84	0.64	5.48	0.26	62	48	22	27	3	27	11	62
1926	58 676	95 202	532	......	......	......	0.36	1.20	1.12	1.23	0.84	0.48	0.09	......	......	5.32	0.62	5.94	0.28	62	46	24	28	2	23	10	67

1 Climatological records at Montrose, Colo.

2 Not recorded.

3 Additional area irrigated from private canals contracted to be turned over to the U. S. Bureau of Reclamation and supplied with Gunnison water: 1912, 4513 A; 1913, 6054 A; 1914, 6000 A; 1915, 4500 A; 1916, 700 A; 1917-23, 1000 A.

4 Project charges based on these deliveries; actual deliveries estimated 5% greater.

5 Prior to 1923 water delivered on continuous flow basis. For 1923 and subsequent years water delivered on demand basis.

6 Total diversions from Uncompahgre and Gunnison Rivers less Gunnison water delivered to Uncompahgre River.

7 Losses small because of unmeasured seepage and farm waste entering canals and laterals. Some return flow charged to canals.

8 Actual losses estimated to average 3% less than shown, due to under-measurement of deliveries.

9 Waste included.

10 Actual deliveries estimated to average 3% more than shown, due to under-measurement.

11 Final irrigable area, 97 064 acres.

TABLE 22.—USE OF WATER ON THE SUNNYSIDE DIVISION OF THE YAKIMA IRRIGATION PROJECT, WASHINGTON.
(Average elevation, 800 ft.; in sand and volcanic ash; lined sections totaling 108 miles in 1915 and 158 miles in 1924.)

Calendar year.	Irrigated area, total for year.[2]	Area commanded by constructed canals.[10]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[4 5] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March, preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season. Canal and lateral losses.[7 8]	Waste.	Delivered to farms.[9]
1912	62 800	80 076	525	......	......	......	0.26	0.52	0.55	0.59	0.55	0.34	0.26	......	......	3.07	0.42	3.49	0.23	60	51	1	20	28	....	....	....
1913	64 400	80 658	525	......	......	......	0.29	0.54	0.49	0.58	0.60	0.43	0.25	......	......	3.18	0.28	3.46	0.20	61	55	....	19	26	....	....	....
1914	66 525	81 807	525	......	......	......	0.29	0.55	0.53	0.58	0.57	0.37	0.18	......	......	3.07	0.41	3.48	0.38	61	54	1	19	26	....	....	....
1915	68 840	89 258	572	......	......	......	0.29	0.48	0.54	0.54	0.26	0.11	0.23	......	3	2.45	0.25	2.70	0.29	63	51	4	24	21	31	7	62
1916	78 000	93 226	585	......	......	......	0.29	0.56	0.56	0.58	0.63	0.43	0.24	......	......	3.29	0.20	3.49	0.52	61	55	3	21	21	26	5	69
1917	80 500	97 285	590	......	......	......	0.16	0.52	0.52	0.65	0.63	0.41	0.27	......	......	3.16	0.17	3.33	0.21	62	51	6	26	17	27	7	66
1918	84 650	98 537	590	......	......	......	0.34	0.60	0.59	0.63	0.61	0.44	0.22	......	......	3.43	0.24	3.67	0.35	64	57	4	24	15	23	7	70
1919	90 000	100 130	605	......	......	......	0.33	0.59	0.57	0.61	0.60	0.36	0.22	......	......	3.28	0.14	3.42	0.23	61	62	5	18	15	24	6	69
1920	93 610	100 783	605	......	......	......	0.26	0.58	0.53	0.51	0.59	0.37	0.20	......	......	3.04	0.35	3.39	0.23	61	65	7	14	14	24	7	69
1921	94 500	101 509	605	......	......	......	0.33	0.59	0.55	0.60	0.59	0.41	0.21	......	......	3.28	0.19	3.47	0.32	62	60	7	18	15	22	7	71
1922	95 000	101 339	605	......	......	......	0.15	0.58	0.59	0.63	0.60	0.42	0.21	......	......	3.18	0.17	3.35	0.36	63	55	6	23	16	23	6	71
1923	95 000	101 329	605	......	......	......	0.36	0.61	0.56	0.61	0.60	0.44	0.12	......	......	3.30	0.40	3.70	0.19	63	57	7	20	16	21	7	72
1924	95 000	102 346	605	......	......	0.02	0.46	0.59	0.62	0.63	0.61	0.34	0.11	......	......	3.38	0.07	3.45	0.21	65	58	6	19	17	22	6	72
1925	95 000	102 335	605	......	......	0.02	0.44	0.62	0.62	0.65	0.63	0.42	0.12	......	......	3.52	0.28	3.80	0.36	63	54	8	22	16	21	6	73
1926	94 000	102 464	605	......	......	0.08	0.52	0.63	0.57	0.63	0.55	0.24	0.07	......	......	3.29	0.15	3.44	0.28	63	54	9	23	14	21	7	72

1 Climatological records at Sunnyside, Wash.

2 Irrigation districts included, 1916 and subsequent years.

3 Shortage of storage water.

4 Deliveries are not generally measured at farms. Lateral losses have been determined to average 15% and the indicated deliveries are 85% of quantities flowing at heads of laterals where water is always measured.

5 Project charges based on these deliveries; actual deliveries estimated as 5% greater.

6 Slight shortage.

7 Lateral losses assumed 15% of discharge at heads of laterals; canals receive unmeasured inflow of waste and seepage water, which has gradually increased in recent years.

8 Actual losses estimated to average 4% less than shown, due to under-measurement of deliveries.

9 Actual deliveries estimated to average 4% more than shown, due to under-measurement.

10 Final irrigable area, 107 600 acres.

TABLE 23.—USE OF WATER ON THE TIETON DIVISION OF THE YAKIMA IRRIGATION PROJECT, WASHINGTON.

(Average elevation, 1 500 ft.; in sandy loam and gravel; lined sections totaling 80 miles in 1913 and 92 miles in 1924.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[3] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, April to September, in feet.[1]	Water delivered plus precipitation, April to September, in feet.	Precipitation, October to March preceding, in feet.[1]	Mean temperature, April to September, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season. Canal and lateral losses.[4]	Waste.	Delivered to farms.[5]
1912	15 008	....	260	......	......	......	......	0.41	0.56	0.51	0.57	0.24	......	......	......	2.29	0.28	2.57	0.50	60	29	6	35	30	....	....	....
1913	18 750	....	335	......	......	......	......	0.45	0.44	0.57	0.55	0.26	......	......	......	2.27	0.25	2.52	0.38	62	33	9	25	33	....	....	....
1914	20 600	30 000	335	......	......	......	......	0.46	0.34	0.54	0.53	0.22	......	......	......	2.09	0.36	2.45	0.54	62	36	12	30	22	27	9	64
1915	22 000	30 000	335	......	......	......	......	0.31	0.47	0.51	0.43	0.12	......	......	[2]	1.84	0.28	2.12	0.57	63	41	18	19	22	27	8	65
1916	23 000	30 000	335	......	......	......	......	0.43	0.46	0.42	0.52	0.32	......	......	......	2.15	0.21	2.36	0.76	60	46	20	13	21	27	7	66
1917	25 400	31 000	335	......	......	......	......	0.43	0.48	0.50	0.50	0.34	......	......	......	2.25	0.13	2.38	0.31	61	46	18	14	22	26	3	71
1918	26 400	31 000	335	......	......	......	0.04	0.47	0.50	0.53	0.51	0.38	......	......	......	2.43	0.12	2.55	0.52	63	45	19	11	25	28	1	71
1919	27 000	32 000	335	......	......	......	0.14	0.54	0.50	0.54	0.53	0.37	......	......	......	2.62	0.10	2.72	0.43	62	52	12	10	26	27	1	72
1920	28 000	32 000	335	......	......	......	0.09	0.52	0.50	0.51	0.50	0.36	......	......	......	2.48	0.21	2.69	0.27	60	52	15	7	26	27	1	72
1921	28 500	32 000	335	......	......	......	0.05	0.49	0.49	0.54	0.54	0.37	0.02	......	......	2.50	0.15	2.65	0.64	60	50	12	7	31	25	4	71
1922	28 700	32 000	335	......	......	......	......	0.46	0.50	0.55	0.55	0.41	0.01	......	......	2.48	0.05	2.53	0.53	63	45	10	9	36	21	3	76
1923	28 350	32 000	335	......	......	......	0.06	0.55	0.48	0.49	0.55	0.41	0.01	......	......	2 55	0.40	2.95	0.47	62	44	9	10	37	21	4	75
1924	27 970	32 000	335	......	......	......	0.13	0.58	0.55	0.56	0.55	0.27	......	......	......	2.64	0.10	2.74	0.36	63	43	6	11	40	21	1	78
1925	27 650	32 000	335	......	......	......	0.10	0.52	0.51	0 55	0.54	0.32	......	......	......	2.54	0.28	2.82	0.49	63	32	11	14	43	25	3	72
1926	28 100	32 000	335	......	......	......	0.29	0.53	0.52	0.54	0.54	0.22	......	......	......	2.64	0.06	2.70	0.41	62	34	9	14	43	22	1	77

[1] Climatological records at Cowiche, Wash.

[2] Water shortage.

[3] Project charges based on these deliveries; actual deliveries estimated to average 5% greater.

[4] Actual losses estimated to average 4% less than shown, due to under-measurement of deliveries.

[5] Actual deliveries estimated to average 4% more than shown, due to under-measurement.

[6] Final irrigable area, 32 000 acres.

TABLE 24.—USE OF WATER ON THE YUMA IRRIGATION PROJECT OF ARIZONA AND CALIFORNIA.

(Average elevation, 120 ft.; in fine sandy loam.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[2] January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.	Precipitation, January to December, in feet.[1]	Water delivered plus precipitation, January to December, in feet.	Total annual precipitation, in feet.[1]	Mean temperature, January to December, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops. Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Percentage of Total Diversions for Season. Canal and lateral losses.[3, 4]	Waste.[3]	Delivered to farms.[5]
1912	13 767	....	171	0.19	0.41	0.36	0.39	0.44	0.41	0.50	0.52	0.51	0.34	0.22	0.21	4.60	0.26	4.86	0.26	71	69	14	17	....	....	....	....
1913	19 607	....	227	0.07	0.31	0.38	0.42	0.43	0.53	0.61	0.51	0.46	0.29	0.16	0.17	4.36	0.09	4.45	0.09	71	74	9	17	....	12	0	25
1914	25 207	....	250	0.20	0.14	0.34	0.33	0.37	0.44	0.44	0.45	0.49	0.26	0.10	0.07	3.63	0.31	3.94	0.31	73	70	9	21	....	....	....	....
1915	27 857	....	307	0.10	0.03	0.40	0.33	0.27	0.44	0.41	0.42	0.40	0.26	0.21	0.14	3.41	0.36	3.77	0.36	71	64	10	26	....	12	25	22
1916	29 483	....	316	0.07	0.12	0.28	0.31	0.33	0.45	0.39	0.48	0.36	0.22	0.12	0.07	3.20	0.20	3.40	0.20	72	59	6	35	....	19	22	24
1917	36 956	....	335	0.06	0.20	0.37	0.36	0.42	0.44	0.46	0.48	0.43	0.28	0.09	0.11	3.70	0.19	3.89	0.19	72	42	2	56	....	19	26	30
1918	45 670	....	338	0.05	0.06	0.31	0.33	0.30	0.40	0.57	0.56	0.42	0.22	0.04	0.03	3.29	0.24	3.53	0.24	72	26	2	72	....	16	20	32
1919	53 284	....	338	0.06	0.11	0.26	0.35	0.25	0.39	0.53	0.44	0.30	0.14	0.04	0.05	2.92	0.17	3.09	0.17	72	30	2	68	....	27	40	33
1920	54 550	....	340	0.04	0.02	0.45	0.28	0.20	0.35	0.59	0.51	0.22	0.16	0.06	0.06	2.94	0.40	3.34	0.40	71	28	1	71	....	10	25	34
1921	52 390	....	345	0.08	0.17	0.31	0.40	0.22	0.29	0.42	0.26	0.24	0.11	0.11	0.08	2.69	0.58	3.27	0.58	72	48	5	47	....	11	23	29
1922	58 970	....	345	0.03	0.14	0.34	0.20	0.23	0.31	0.33	0.35	0.22	0.24	0.12	0.08	2.59	0.25	2.84	0.25	72	54	2	44	....	10	24	26
1923	53 270	....	345	0.11	0.19	0.33	0.31	0.23	0.34	0.34	0.41	0.24	0.24	0.10	0.06	2.90	0.33	3.23	0.33	71	50	4	46	....	16	61	23
1924	53 207	....	324	0.06	0.19	0.36	0.34	0.22	0.44	0.53	0.54	0.33	0.22	0.08	0.08	3.39	0.07	3.46	0.07	72	45	1	54	....	14	59	27
1925	55 904	64 865	324	0.08	0.18	0.39	0.25	0.24	0.45	0.50	0.44	0.29	0.10	0.05	0.08	3.05	0.32	3.37	0.32	72	35	1	64	....	16	57	27
1926	60 290	64 865	324	0.07	0.19	0.38	0.21	0.20	0.41	0.42	0.28	0.15	0.15	0.11	0.02	2.59	0.77	3.36	0.77	73	36	5	59	....	14	63	23

[1] Climatological records at Yuma, Ariz. Weather Bureau Station.

[2] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[3] Losses and waste from lateral system only, for 1912 to 1918, and from 1920 to 1922, loss and waste from main canal being inseparable. Percentage based on diversions from Colorado River at Laguna. Available water supply at all times exceeds project requirement, and it is customary to operate with canals largely filled to minimize silt deposition.

[4] Actual losses estimated as averaging 3% less than shown, due to under-measurement of deliveries.

[5] Percentage expressed in terms of diversions from Colorado River, at Laguna. Actual deliveries estimated as averaging 3% more than shown, due to under-measurement.

[6] Final irrigable area, 108 000 acres.

TABLE 25.—AVERAGE USE OF WATER ON FEDERAL IRRIGATION PROJECTS.

TABLE 25.—AVERAGE USE OF WATER ON FEDERAL IRRIGATION PROJECTS.

Detailed data in Table:	Project.	Irrigated area, in acres.	Area commanded by constructed canal system, in acres.	Miles of canals and laterals operated.	Miles of canals and laterals lined or enclosed.	Predominating character of soils.	Delivered (Charged) to Farms, in Hundredths of Acre-Feet per Acre.[10]						
							January.	February.	March.	April.	May.	June.	July.
1	Belle Fourche	45 164	74 569	547	58	Heavy	......	......	......	......	0.07	0.25	0.40
2	Boise[1]	145 616	163 667	1 004	37	Light	......	......	......	0.16	0.73	0.78	0.89
3	Carlsbad	22 535	25 000	45	11	Medium	0.04	0.06	0.09	0.51	0.29	0.28	0.37
4	Grand Valley[2]	10 139	29 000	180	7	Heavy	......	......	......	0.14	0.69	0.78	0.81
5	Huntley	19 406	32 540	232	......	Heavy	......	......	......	......	0.12	0.28	0.51
6	King Hill[3]	6 460	16 890	96	43	Very light	......	......	......	0.31	1.23	1.30	1.51
7	Klamath	43 325	52 396	240	2	Medium	......	......	......	0.01	0.31	0.41	0.35
8	Lower Yellowstone	17 540	48 272	202	......	Heavy	......	......	......	......	0.07	0.30	0.51
9	Milk River	16 793	63 480	275	......	Heavy	......	......	......	0.01	0.11	0.22	0.21
10	Minidoka-South Side Pumping Division	44 945	48 880	275	......	Medium	......	......	......	0.06	0.41	0.56	0.66
11	Newlands	38 808	65 277	319	......	Medium	......	......	0.08	0.28	0.58	0.53	0.64
12	North Platte	107 694	161 870	1 154	......	Medium	......	......	......	......	0.12	0.41	0.67
13	Okanogan[4]	5 260	7 300	68	39	Light	......	......	......	......	0.37	0.42	0.68
14	Orland[5]	14 554	20 600	135	89	Light	0.01	0.02	0.11	0.27	0.54	0.57	0.63
15	Rio Grande[6]	96 847	126 300	485	10	Medium	......	0.04	0.18	0.41	0.47	0.49	0.48
	Shoshone:												
16	Frannie Division[7]	7 963	20 063	166	......	Heavy	......	......	......	0.02	0.29	0.56	0.68
17	Garland Division[8]	32 380	42 000	279	4	Medium	......	......	......	0.05	0.30	0.57	0.71
	Sun River:												
18	Fort Shaw Division	7 650	13 902	99	......	Heavy	......	......	......	......	0.16	0.45	0.52
19	Greenfields Division[9]	9 867	41 975	190	......	Medium	......	......	......	......	0.08	0.35	0.65
20	Umatilla[2]	10 970	24 587	173	157	Light	......	......	0.02	0.52	0.98	0.96	1.07
21	Uncompahgre	61 178	95 202	470	11	Medium	......	......	......	0.39	1.11	1.18	1.21
	Yakima:												
22	Sunnyside Division	91 726	102 464	602	125	Medium	......	......	0.01	0.33	0.59	0.57	0.62
23	Tieton Division	27 607	32 000	335	86	Light	......	......	......	0.09	0.51	0.50	0.58
24	Yuma	51 950	64 865	336	......	Medium	0.06	0.15	0.35	0.30	0.25	0.38	0.47

[1] 1924 and 1926 omitted from average on account of water shortages.
[2] Umatilla and Grand Valley data cover years, 1916-25.
[3] King Hill data cover years, 1921-27.
[4] Okanogan data average for 1921, 1923, and 1925, on account of shortages in all other years since 1917.
[5] Orland data omit years of heavy water shortage, 1918, 1920, and 1924.
[6] Rio Grande data cover years, 1919-26.
[7] Shoshone-Frannie Division data cover years, 1922-26.
[8] Losses on water conveyed through Garland Division Main Canals for Frannie Division included in Garland Division losses.
[9] 1919 to 1926, inclusive.
[10] Data are for years, 1917 to 1926, inclusive, except as noted.

TABLE 25.—(*Continued*).

Detailed data in Table:	Project.	Delivered (Charged) to Farms, in Hundredths of Acre-Feet per Acre.[10]						Precipitation, in growing season, in feet.	Water delivered plus precipitation in growing season, in feet.	Precipitation, preceding growing season, in feet.	Elevation of project, in feet.	Mean temperature in growing season, in degrees Fahrenheit.	Percentage of Area in Different Crops.				Percentage of Total Diversions.		
		August.	September.	October.	November.	December.	Total.						Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.	Delivered to farms.
1	Belle Fourche	0.35	0.15	......	......	......	1.22	0.86	2.08	0.48	2 800	65	62	22	16	....	33	15	52
2	Boise[1]	0.71	0.29	0.04	......	......	3.60	0.38	3.98	0.43	2 500	62	53	31	13	3	28	2	70
3	Carlsbad	0.44	0.20	0.06	0.02	......	2.36	0.86	3.22	0.06	3 100	66	33	4	63	....	48	6	46
4	Grand Valley[2]	0.61	0.35	0.19	0.04	......	3.61	0.48	4.09	0.27	4 700	65	36	24	33	7	43	22	35
5	Huntley	0.37	0.11	......	......	......	1.39	0.63	2.02	0.37	3 000	64	42	34	24	....	36	30	34
6	King Hill[3]	1.48	0.97	0.20	0.01	......	7.01	0.35	7.36	0.41	2 750	64	75	7	11	6	......	....	53
7	Klamath	0.30	0.05	......	......	......	1.43	0.18	1.61	0.68	4 100	60	77	21	2	....	39	9	52
8	Lower Yellowstone	0.35	0.11	......	......	......	1.34	0.71	2.05	0.33	1 900	64	45	33	22	....	44	21	35
9	Milk River	0.07	0.02	0.01	......	......	0.65	0.91	1.56	0.18	2 200	58	73	22	5	....	36	19	45
10	Minidoka-South Side Pumping Division	0.56	0.27	0.02	......	......	2.54	0.53	3.07	0.32	4 200	59	50	28	22	....	39	3	58
11	Newlands	0.46	0.28	0.06	0.02	......	2.88	0.25	3.13	0.15	4 000	59	85	11	4	....	41	14	45
12	North Platte	0.65	0.38	......	......	......	2.23	0.81	3.04	0.47	4 100	66	36	26	38	....	43	8	49
13	Okanogan[4]	0.68	0.45	......	......	......	2.60	0.42	3.02	0.51	1 000	63	9	....	3	88	29	....	71
14	Orland[5]	0.57	0.40	0.04	0.01	......	3.17	0.37	3.55	1.02	250	70	53	5	19	23	27	9	64
15	Rio Grande[6]	0.42	0.30	0.10	0.03	0.02	2.89	0.60	3.49	0.06	3 700	66	37	8	53	2	32	39	29
	Shoshone:																		
16	Frannie Division[7]	0.46	0.19	0.04	......	......	2.19	0.39	2.58	0.11	4 150	60	72	17	11	....	42	21	37
17	Garland Division[8]	0.50	0.21	0.03	0.01	......	2.38	0.33	2.71	0.09	4 400	59	59	28	13	....	38	7	55
	Sun River:																		
18	Fort Shaw Division	0.36	0.04	0.01	......	......	1.54	0.60	2.14	0.23	3 700	58	75	20	5	....	36	26	38
19	Greenfields Division[9]	0.13	0.01	0.06	......	......	1.28	0.65	1.93	0.28	3 700	57	23	75	2	....	31	22	47
20	Umatilla[2]	0.96	0.44	0.06	0.01	......	5.02	0.35	5.37	0.40	470	60	85	1	6	8	32	18	50
21	Uncompahgre	0.92	0.63	0.32	......	......	5.76	0.55	6.31	0.26	5 500	61	47	25	25	3	13	10	77
	Yakima:																		
22	Sunnyside Division	0.60	0.39	0.18	......	......	3.29	0.21	3.50	0.27	800	63	57	7	21	15	23	7	70
23	Tieton Division	0.53	0.35	......	......	......	2.51	0.16	2.67	0.44	1 500	62	44	12	11	33	24	2	74
24	Yuma	0.43	0.28	0.19	0.08	0.07	3.01	0.33	3.34	0.33	120	72	39	3	58	....	14	58	28

[1] 1924 and 1926 omitted from average on account of water shortages.
[2] Umatilla and Grand Valley data cover years, 1916-25.
[3] King Hill data cover years, 1921-27.
[4] Okanogan data average for 1921, 1923, and 1925, on account of shortages in all other years since 1917.
[5] Orland data omit years of heavy water shortage, 1918, 1920, and 1924.
[6] Rio Grande data cover years, 1919-26.
[7] Shoshone-Frannie Division data cover years, 1922-26.
[8] Losses on water conveyed through Garland Division Main Canals for Frannie Division included in Garland Division losses.
[9] 1919 to 1926 inclusive.
[10] Data are for years, 1917 to 1926, inclusive, except as noted.

Methods of Measurement

Reliable gauging stations are maintained at the diversion points on all projects. Discharge measurements at the heads of laterals are often complicated by canal structures. Waste, as the term is used in this compilation, is the water returned to natural streams through wasteways operated in connection with the project system. It does not include water delivered to the user and by him permitted to leave his farm unused. Project waste is measured by the project operation forces. The data on waste and losses are made to portray conditions without internal reservoirs by eliminating the reservoir losses where such reservoirs are present.

Farm deliveries are ascertained in a number of ways and with varying degrees of accuracy. In general, they are measured at, or very close to, the farm border; but in a few cases complications attending such observations have resulted in a practice of making measurements nearer the main canals with allowance for losses in transit to the farm border. The most common form of measurement is by means of the Cipolletti weir. However, a lack of adequate heads often precludes this method, so that the use of submerged weirs or current meters becomes necessary. The cost of current-meter work precludes the securing of accurate delivery records where this method is required. The use of the current-meter method is practically limited to parts of the Rio Grande and Yuma Projects.

In addition to the difficulties of water measurement, the procurement of records truly reflecting the use of water is complicated on many projects by human factors, because the ditch rider is constantly besieged by individuals to reduce their water charges. On many projects there is, therefore, a recognized factor of under-measurement of farm deliveries, with a corresponding increase in apparent canal and lateral losses, because diverted waters not accounted for in waste and deliveries, are charged to canal loss. At numerous times and places, measurements have been made to ascertain the loss in feet of depth per day for various types of canal materials, but the utilization of these results for an estimate of total canal losses in an operating system is impractical, by reason of cost and changing conditions.

Meaning of the Data

Referring to the tabulations for the individual projects, the area commanded by the constructed canal system represents the irrigable area to which water could have been supplied. The irrigable area, with minor deviations on various projects, excludes rights of way for section-line roads, highways, railroads, main canals, principal laterals, and drains. It also excludes all lands that cannot be cultivated at moderate costs for land preparation and farm lateral construction or lands that are unworthy of cultivation by reason of inferior soils. House and barn yards have not been excluded. Areas that are temporarily non-irrigable by reason of seepage are included. Declining areas are usually due to elimination of lands on account of inferior agricultural values or on account of permanent seepage. The lag in the irrigated area is due to delayed settlement and delayed development of individual holdings.

The comparison of irrigated and commanded lands, together with data on the character of canal construction and length of canals, are given as a guide for the interpretation of canal losses and project waste which can usually be expected to decline as full development is approached. In many cases the main canals have been constructed to a capacity that is considered adequate for the ultimate irrigable area, so that their operation to partial capacity is productive of undue losses.

The farm deliveries represent, in general, the quantities for which charges are made. On a few projects, "off-season" deliveries are customarily made to supply stock water and on others the use of "off-season" water is at times encouraged for leaching purposes and for conserving the limited supplies available during the irrigation season. The general practice is to make a small charge, or no charge at all, for "off-season" deliveries. The tabulation excludes such deliveries and the attendant losses, in so far as possible without research into the original water records. The column headed, "Actual Delivery Percentage Larger", represents the required increase necessary in the tabulated farm deliveries to ascertain actual deliveries, on account of the recognized factor of under-measurement. The indicated correction factor to ascertain actual deliveries has been supplied by the operating personnel. This is necessarily an opinion only, but it is supported by long experience and familiarity with project conditions, and in most cases by numerous although discontinuous measurements for the purpose.

Data on temperature, precipitation, character of crops, and character of soils, have been added to aid in interpreting the results. The growing season mentioned comprises the months in which, if rainfall were inadequate, it would be necessary to resort to irrigation. As a further guide, the precipitation during the non-growing months preceding the growing season also has been shown, because in many cases it replenishes soil moisture used in the growing season.

Alfalfa and forage crops, together with irrigated pastures, are heavily irrigated throughout the season—as a rule, by surface flooding. Fields of small grains, although usually irrigated by flooding, require the least total water because the growing period during the irrigation season is short and plant requirements are relatively small. Furrow crops, of which potatoes, sugar-beets, corn, and small fruits are predominant, require more water than small grains, but generally less than the forage crops. Trees, usually fruit orchards, have a low water requirement except when intercropped and then such requirement is largely determined by the character of the intercropping.

Canal and lateral losses do not include losses in canals used solely as reservoir feeders. A case of this type is the Hermiston Division of the Umatilla Project (Table 20), the losses in the Echo Feed Canal being disregarded. Where irrigation deliveries are made both above and below reservoirs served by project canals, as on the Interstate Division of the North Platte Project (Table 12), no attempt to correct for canal losses on water subsequently lost by evaporation and seepage has been made. Reservoir losses by evaporation and seepage are not included in the canal and lateral losses. Where the available data permit, the diversions include—in addition to waters from primary sources of supply, such as rivers and reservoirs—waters diverted from artificial

and natural drainage courses within the project, although these may have been included once before with waters diverted from primary sources of supply.

PROJECT NOTES

Special conditions on the projects and their effect in the submitted data may be mentioned as follows.

Belle Fourche Project.—The climate is semi-arid and many farmers irrigate only a part of the area to which water is delivered, thereby making it impossible to determine the actual irrigated areas, except at considerable cost. This practice is gradually being abandoned as increasing irrigation charges necessitate better farming. (See Table 1.)

Boise Project.—A considerable area of this project is dependent on Deerflat Reservoir which is filled through the project main canal when its capacity is not needed for the irrigation of lands above the reservoir. (See Table 2.) The reservoir also receives much waste water and some seepage from the higher lands. For some years records of such inflow have not been obtained. The data presented do not include losses from the main canal, which, from data obtained in earlier years, appear to average approximately 5% of the diversions.

Carlsbad Project.—The storage facilities are inadequate to control stream flow during the non-irrigation season. (See Table 3.) The stream flow during the irrigation season, together with impounded waters, is often less than desired. Pre-season deliveries are, therefore, prevalent.

King Hill Project.—A uniform supply of water is available throughout the season, and while it is inadequate for the irrigation of the entire project, it permits extensive waste at the beginning and end of the irrigation season. (See Table 6.)

Newlands Project.—A limited water supply in 1924 and 1926 caused a material reduction in project waste as compared with earlier years when elimination of such waste was not necessary and the wasting of water was permitted to continue in the interest of superior water service at lowest cost. Recent general increases in farm deliveries are in part traceable to drainage construction. (See Table 11.)

Rio Grande Project.—The apparent great waste of water on this project results from its physical arrangement. (See Table 15.) The project consists of three successive valleys served by six diversions from the Rio Grande. It is customary to run canals at almost maximum capacity in the upper valleys through a large part of the irrigation season with surplus water wasted back to the stream and re-diverted at lower points.

Shoshone Project.—Waters for the Frannie Division (Table 16) are first carried through the main canals of the Garland Division (Table 17), and the resultant losses are included with local losses on the Garland Division. Drainage construction under way on the Garland Division may be expected to result in augmented farm deliveries.

Sun River Project.—The Greenfields Division (Table 19) has a soil and climate adapted to the production of grain by dry-farming methods. Lack of adequate storage reserves has deterred initiation of the collection of construction charges while operation and maintenance charges have been collected only

when water was requested and used. The result has been a haphazard farming practice with little reliance on irrigation. With the completion of the Gibson Reservoir now under construction, it is expected that irrigation will be placed on a more stable basis.

Umatilla Project.—The water supply for the East Division of this project, comprising more than one-half the project area, is furnished by Cold Springs Reservoir, which is served by a feed canal from Umatilla River. The indicated canal losses (Table 20) do not include feed-canal losses, which average about 9 per cent.

Yakima Project.—On the Tieton Division (Table 23), the main canal, which is largely concrete flume or concrete-lined, lacks sufficient capacity to meet project demands at the height of the irrigation season. The maximum rate of delivery is, therefore, unduly prolonged. Corresponding data for the Sunnyside Division are given in Table 22.

Yuma Project.—The heavy waste of water on this project (see Table 24) is in part occasioned by the operation of a power plant within the canal system from which the used water is returned to the river when irrigation requirements are small. The main canal is usually operated at near maximum levels in order to minimize silt deposition and the necessity for frequent sluicing. Conservation of water is no object except for occasional short periods when the requirements of the Imperial Valley Canal, diverting below the power-plant return, make the return of all project waste above the Imperial Intake desirable.

The summary compilation in Table 25 gives the average use of water during the past ten years in which water supply was fully adequate. In addition, it classifies the projects as to their predominant soil types. It should not be inferred, however, that the indicated soil type is present to the practical exclusion of others, as, in fact, the soils present in each project in almost every case vary from some form of clay to sand. Were this condition not present, the indicated variation in water use would be even greater than that shown.

SUMMARY

In this compilation, the quantities shown on the individual sheets have been modified in the case of the Belle Fourche, Huntley, Klamath and Rio Grande Projects to correspond with an undermeasurement of 5% in delivery to farms. With these changes, the farm deliveries shown on the summary sheet (Table 25) are then believed to be from zero to 10% less than the actual delivery, and the losses are actually correspondingly less.

The indicated use of water may be considered to represent closely the maximum quantities that the irrigator can use advantageously. Financial considerations have had little influence in engendering a high duty of water because the construction repayment burden on Federal projects is relatively light. On most projects the lands now in cultivation are prone to use waters more freely than will be possible with the project fully developed. Only the Okanogan and Orland Projects have experienced shortages of sufficient severity to influence development. On the whole, it is probable that a higher duty of

water prevails on private projects where financial considerations exert a strong influence on the canal and reservoir capacities provided.

Continuous efforts are being made to reduce the excess quantities of water delivered to the farms as well as the transmission losses. However, in practically all cases both excesses are returned to the streams, either as surface run-off or drain discharges, at points such that the unused water is available for re-diversion and beneficial use by other irrigators.

The data indicate project waste and canal losses that may appear surprisingly large to many engineers. Canal losses join the underlying water-table and return to natural channels. Wasted waters so return on the surface. With minor exceptions such return flow is utilized by other projects with a resulting small net loss of waters. Where physical conditions are favorable for project extension, increasing land values in time will foster higher water duties through elimination of waste and reduction in canal losses and farm deliveries. Drainage problems will thereby be minimized and soil fertility conserved with extensive benefits unless carried to the extreme of an insufficient application of water to prevent injurious accumulations of deleterious salts. Reduction in waste, use, and losses will be accompanied by a reduction in return flow with increasing salt content to the detriment of projects that may have become dependent on such supplies.

Much credit is due the various project superintendents for their aid in the preparation of the data and to the Assistant Engineers, C. C. Elder, Assoc. M. Am. Soc. C. E., and Mr. T. R. Smith, to whom much of the work was entrusted and who, by their unflagging interest, have made the completion of this compilation possible. The operation and maintenance of Federal irrigation projects, as well as all engineering and construction work, is under the direction of R. F. Walter, M. Am. Soc. C. E., Chief Engineer of the U. S. Bureau of Reclamation, with headquarters at Denver, Colo.; and all activities of the Bureau are under the general charge of Elwood Mead, M. Am. Soc. C. E., Commissioner, with headquarters at Washington, D. C.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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STREAM FLOW IN GENERAL TERMS

Discussion*

By Melvin D. Casler, M. Am. Soc. C. E.†

Melvin D. Casler,‡ M. Am. Soc. C. E. (by letter).§—The discussions brought forth by this paper are all very helpful in focusing attention to the fact that the consideration of "stream flow in general terms" means much more than the simple appearance of the Chezy formula would indicate.

Mr. Bailey‖ makes the very pertinent observation that it is often of greatest importance to appreciate that a given quantity of water with a given energy content, $d + h$, can flow at two different depths in a given channel.

He refers to the case of an entering or discharging stream at an intersection with a branch channel. In this case, as regards sections above and below the intersection, Q_x and Q_y are unequal, and the following treatment is suggested:

Let Q_x' and Q_x'' represent the respective flows of two entering streams; Q_y' and Q_y'', the respective flows of two departing streams; and, Q_x or Q_y, as the case may be, the flow at the junction. Other factors at Sections x and y are similarly indicated. I_1 and F_1 denote invert drop and total head loss, respectively, between the "primed" sections and the stream junction, while I_2 and F_2 denote the same between "double-primed" sections and the junction. Then, in the case of converging flow, $Q_x' + Q_x'' = Q_y$, and for diverging flow, $Q_x = Q_y' + Q_y''$. Following the reasoning used in the analysis of Fig. 1¶:

For converging flow,

$$W_x' (d_x' + I_1) + W_x' h_x' + W_x'' (d_x'' + I_2) + W_x'' h_x'' = W_y (d_y + h_y) + W_x' F_1 + W_x'' F_2.$$

* Discussion of the paper by Melvin D. Casler, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Author's closure.

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§ Received by the Secretary, December 11, 1928.

‖ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1605.

¶ *Loc. cit.*, January, 1928, Papers and Discussions, p. 100.

Replacing each W by its corresponding $w\,Q$:

$$(d_y + h_y) = \frac{Q_x'}{Q_y}(d_x' + h_x' + I_1 - F_1) + \frac{Q_x''}{Q_y}(d_x'' + h_x'' + I_2 - F_2)$$

in which,

$$Q_y = Q_x' + Q_x''$$

Likewise for diverging flow,

$$(d_x + h_x) = \frac{Q_y'}{Q_x}(d_y' + h_y' + F_1 - I_1) + \frac{Q_y''}{Q_x}(d_y'' + h_y'' + F_2 - I_2)$$

in which,

$$Q_x = Q_y' + Q_y''$$

Additional terms of like form may be appended to these equations to provide for three or more tributaries or branches with a common junction.

As stated by Mr. Pearce* the paper presents nothing basically new. Its prime purpose is to facilitate the analysis of stream flow in irregular channels and to present a direct method of treatment applicable to all conditions, capable of being understood and applied by the average computer, and sufficiently short to permit the inclusion of certain generally accepted principles that are usually ignored because of the laborious calculations involved in their application by current methods—most of which sacrifice accuracy for simplicity, or *vice versa.* Most existing literature on this subject is either too vague in its generalities or else it is directly applicable only to certain special cases.

As suggested by Mr. Mavis,† unless the significance of the friction factor, f, is clearly recognized there may be danger of confusing the writer's f with the factor, f, in the familiar pipe formula, $h' = f\,\frac{l\,v^2}{d\,2\,g}$, in which, h' is the head loss in the length, l, due to perimeter friction, and d is the diameter of the pipe. In the writer's nomenclature, $h' = f\,l$, and, therefore, the writer's f equals the f of the pipe formula multiplied by $\frac{h}{d}$, in which, h is the velocity head, $\frac{v^2}{2\,g}$, and d is the pipe diameter.

With reference to Mr. Mavis' subsequent discussion it must be borne in mind that in the basic Equations (10)‡ and (11),‡ F is the total aggregate head loss between Sections x and y, due to all causes of whatever nature. The factor, f, is the unit head loss, due to perimeter friction only, and $F = \int_0^l f\,d\,l + F'$, in which, F' is the total head loss between x and y due to internal friction (eddies and impact due to sudden internal changes in velocity and direction). Equations (10) and (11), as stated, are theoretically correct and comprehensive and are applicable alike to one, two, or three-dimensional flows as long as Q is constant; that is, if no water enters or leaves the stream between Sections x and y.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1606.

† *Loc. cit.*, August, 1928, Papers and Discussions, p. 1899.

‡ *Loc. cit.*, January, 1928, Papers and Discussions, p. 101.

The great obstacle in the way of the direct application of Equations (10) and (11) is the evaluation of the internal losses, F', which must include the losses incident to two or three-dimensional flows. The factor, F', is "the goat", so to speak, and must carry losses of sufficient aggregate magnitude to bridge all apparent discrepancies between Equations (10) and (11) and any and all observed hydraulic phenomena and experimental deductions.

At a hydraulic jump, Sections x and y are coincident and are operating at low stage and high stage, respectively; $l = 0$, $I = 0$, $F = F' =$ the head loss due to the jump, and, from Equation (10), $(d_x + h_x) - (d_y + h_y) = F'$; which refutes Mr. Mavis' suggestion that Criterion (3) would lead to the impression that $(d + h)$ is the same before and after the jump. Mr. Mavis overlooks the words, "as nearly as possible", in the criterion.

In the illustrative example plotted in Fig. 6,* F' was ignored and, in the treatment of weirs, the entire, F, between Sections x and y was assumed as zero.

It is interesting to note that in a problem like that of Fig. 6, in which the channel is of constant cross-section, analyzed in stretches of uniform length for perimeter friction, or for any losses directly proportional to l, it is possible to eliminate all "cut and try" features by the use of a double curve as illustrated in Fig. 18. Assuming that $F = l$ times the average f between Sections x and y:

$$F = \frac{l}{2}(f_x + f_y) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (98)$$

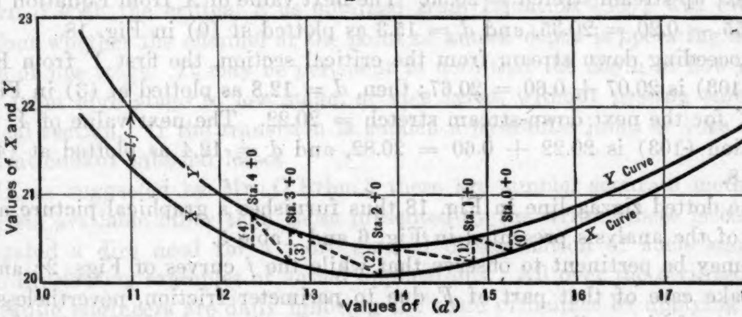

FIG. 18.

and, from the basic Equation (6),†

$$d_x + h_x + I = d_y + h_y + \frac{l}{2} f_x + \frac{l}{2} f_y \ldots\ldots\ldots (99)$$

$$d_x + h_x - \frac{l}{2} f_x = d_y + h_y + \frac{l}{2} f_y - I \ldots\ldots\ldots (100)$$

$$d_y + h_y + \frac{l}{2} f_y = d_x + h_x - \frac{l}{2} f_x + I \ldots\ldots\ldots (101)$$

Let $X = d_x + h_x - \frac{l}{2} f_x$, and, $Y = d_y + h_y + \frac{l}{2} f_y$. Then,

$$X = Y - I \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (102)$$

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 110.

† *Loc. cit.*, p. 100.

proceeding up stream, and, proceeding down stream,

$$Y = X + I \qquad (103)$$

The problem of Fig. 6 may now be analyzed as illustrated in Table 16.

TABLE 16.—HYDRAULIC ELEMENTS.

(Q = 6 000 sec-ft. l = 100 ft.)

d.	$h = m^* Q^2$.	$f = n^* Q^2$.	$X = d + h - \frac{l}{2} f$.	$Y = d + h + \frac{l}{2} f$.
10.469	11.866	0.0078	21.95	22.73
11.625	9.392	0.0058	20.73	21.31
15.000	5.422	0.0030	20.27	20.57
18.000	3.787	0.0019	21.69	21.88

* From *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 108, Table 4.

These values of X and Y are then plotted as shown in Fig. 18. From Table 5,* the value of d at Station 2 + 0 (which is the critical section (see Fig. 6)), is 13.8. From Fig. 18, for d = 13.8, X and Y are 20.07 and 20.44, respectively.

Proceeding up stream, the next value of X from Equation (102) is 20.44 − 0.20 = 20.24, from which, d = 14.85 as plotted at (1) in Fig. 18, and Y for the next up-stream stretch = 20.55. The next value of X from Equation (102) is 20.55 − 0.20 = 20.35, and d = 15.3 as plotted at (0) in Fig. 18.

Proceeding down stream from the critical section, the first Y from Equation (103) is 20.07 + 0.60 = 20.67; then, d = 12.8 as plotted at (3) in Fig. 18 and X for the next down-stream stretch = 20.22. The next value of Y from Equation (103) is 20.22 + 0.60 = 20.82, and d = 12.4 as plotted at (4) in Fig. 18.

The dotted zigzag line in Fig. 18 thus furnishes a graphical picture and a check of the analysis presented in Fig. 6 and Table 5.

It may be pertinent to observe that while the f curves of Figs. 2† and 4‡ only take care of that part of F due to perimeter friction, nevertheless the Q-curves of those diagrams are comprehensive in their application to the solution of Equations (10) and (11) and are just as applicable to the consideration of internal losses or combined losses as they are to the f losses.

Fig. 6 is a case similar to the convex invert slope referred to by Mr. Mavis.§ Whether the water is accelerated or decelerated below the break in grade may be determined by the proper application of Equations (10) and (11).

Regarding Mr. Mavis' reference to the impulse and momentum equations§ it may not be over sanguine to entertain the hope that there may be found ultimately a workable method of introducing those equations in conjunction

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 111. Fig. 18 is seen to uncover an error in the water depth at Station 0+0, which is 15.3 instead of 15.1, as given in the paper.

† *Loc. cit.*, p. 103.

‡ *Loc. cit.*, p. 107.

§ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1900.

with the energy equations in such a way as to relieve the internal friction factor, F', from some of the responsibility which it bears in Equations (10) and (11).

Mr. Mavis states* in his reference to Criterion (4), that one would infer that a critical stage must be found before the calculations can be made. Such an inference was not intended. The computations may start from a critical section or from any other section where there is a condition of known depth for a known flow.†

It should be noted that Criterion (3) refers only to the relations between two successive sections, one of which has already been determined, and that Criterion (4) is merely a re-statement of Criterion (3) as applied to an analysis starting from the critical section. In the general criterion that F will be as nearly as possible equal to I, it is important to remember that F must include the losses due to any sudden changes in stage of flow.

An analysis is often started with an assumed depth at one end of the channel. If, at any stage in the progressive application of Equation (10) or Equation (11), a section is encountered at which the computed value of $(d + h)$ is less than the critical $(d + h)$ of that section for the assumed Q, that section is established as the critical section as regards all sections previously computed, and a new start is made at that section, revising all previous computations in reverse order.

If a known quantity is flowing at a known depth a comparison of the known d with the critical d as obtained from Equation (18)‡ will establish the fact whether the channel at the point of known depth is operating at high stage or low stage. It may be pertinent to note that the depth of flow cannot pass from high stage to low stage, or *vice versa*, without passing through a critical section. If the transition is sudden a hydraulic jump or bore occurs, with attendant internal losses.

If, as suggested by Mr. O'Brien,§ there are simple, accurate methods of analysis available other than those presented by the writer, there is certainly indicated a dire need for a more general dissemination of such knowledge with illustrative examples of actual applications. All over the civilized world hydraulic engineers are daily ignoring the basic principles by applying simple approximations in the belief that more accurate analyses are impractical; or else they are laboriously plodding through reams of "cut and try" computations similar to those cited by Mr. Pearce,‖ who demonstrates the speed and accuracy of the method of attack proposed by the writer. The only irksome feature of the writer's method is the preparation of the curves of Figs. 2 and 4 for the various sections involved. The computation of these curves and their critical factors may be readily standardized, as indicated in Tables 1, 2, 3, and 4, and reduced to systematic treatment. A computing machine is, of course, a great aid in any such operations.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1901.

† *Loc. cit.*, January, 1928, Papers and Discussions, p. 113.

‡ *Loc. cit.*, p. 104.

§ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1902.

‖ *Loc. cit.*, May, 1928, Papers and Discussions, p. 1606.

Mr. O'Brien calls attention* to the erroneous assumption of uniform velocity throughout the cross-section of a stream approaching a weir. This is a real error and is present in nearly all stream-flow calculations. The assumption is as stated in the definition of h in the "Nomenclature"† and is again referred to in the parenthetical note following Equation (13)‡ and the errors thus introduced are recognized and discussed in three places throughout the paper.§ The profession would owe a vote of thanks to any one who presents a practical method of taking due account of non-uniform velocities. If their effect could be evaluated there would be some hope for the universal weir diagram presented in Fig. 8.‖

Mr. Simham¶ sounds a warning against the application of the results of small laboratory experiments to practical cases of large magnitude, without due consideration of the general laws involved.

Practical experimentation is an invaluable aid in establishing coefficients and often calls attention to certain physical laws and limitations and to various practical considerations which would never have occurred to the mathematician in concocting his formulas on a purely academic basis. Nevertheless, before attempting to make general empirical application of deductions drawn from laboratory experiments, every effort should first be made to interpret fully the experiments and deduce, if possible, the basic laws operating to produce the observed results and phenomena.

The quantity, $K\ p\ v^3$, referred to by Mr. Simham, is the power loss (or energy loss per second) per running foot of channel; although the writer must admit that this fact may not be apparent from the paper.

The writer's use of W, or Q, as the water unit in his derivation of the Chezy formula, Equation (1), leads to some confusion, since Q is an appreciable quantity of water spread over an appreciable length of channel throughout which v cannot remain constant under general conditions. The results are the same no matter to what quantity of water the analysis is applied, but the significance of the various factors may be more apparent from the following derivation: Let q represent a minute elemental quantity of water at any particular cross-section of the channel, expressed in cubic feet. The factor, f, is the rate of head loss per foot of channel due to perimeter friction; and hence the energy lost by the quantity of water, q, per foot of forward movement is $w\,q\,f$ ft-lb. (Criterion 1).**

The length of channel occupied by the quantity, q, is $\frac{q}{a}$, and the area of contact between q and the interior of the channel is $\frac{p\,q}{a}$ sq. ft.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1908.

† *Loc. cit.*, January, 1928, Papers and Discussions, p. 98.

‡ *Loc. cit.*, p. 103.

§ *Loc. cit.*, p. 115, 118–119.

‖ *Loc. cit.*, p. 117.

¶ *Loc. cit.*, January, 1929, Papers and Discussions, p. 119.

** *Loc. cit.*, January, 1928, Papers and Discussions, p. 99.

The unit frictional resistance, in pounds per square foot of water contact, is taken as $K v^2$, K being a constant and v being assumed as constant for the infinitesimal channel length, $\frac{q}{a}$, occupied by the quantity, q.

Hence, the total frictional resistance acting against the flow of the quantity of water, q, is $\frac{p\,q}{a} K v^2$ lb.; and the work done by q per ft. of forward movement in overcoming this resistance is $\frac{p\,q}{a} K v^2$ ft-lb. (Criterion 2).*

Equating the work done to energy consumed, Criteria (1) and (2) furnish the following:

$$\frac{p\,q}{a} K v^2 = w\,q\,f \qquad (104)$$

which, when simplified, becomes identical to Equation (1).

Since f is the head loss per linear foot of channel, $w f$ is the energy, in foot-pounds, lost by 1 cu. ft. of water in advancing over 1 lin. ft. of channel; and, since Q = the quantity of water passing, in cubic foot per second, the rate of energy loss at any particular cross-section of the stream, in foot pounds per second per linear foot of channel, is $Q\,w\,f = w\,a\,v\,f$ (assuming the factor, f, to cover all losses).

From Equation (1), $w\,a\,v\,f = K\,p\,v^3$. Therefore, $K\,p\,v^3$, or its equivalent, $\frac{w\,p\,v^3}{C^2}$, is the power loss per linear foot of channel.

The ratio of the power-loss rate to the kinetic power delivered is seen to be $\frac{f}{h} = \frac{n}{m} = \frac{2\,g}{C^2\,r}$.

It would appear that Mr. Simham's equations are incomplete in that they take no account of the progressive changes in the hydraulic elements of the stream section.

In Equation (71)* it is necessary to recognize that the quantities, p and v, represent coincident factors at one particular cross-section of the stream; that is, $K\,p_x\,v_x^3$ is the rate of power loss at Section x and $K\,p_y\,v_y^3$ is the rate of power loss at Section y. Hence, the total power loss between Sections x and y is,

$$\frac{K\,l}{2}\left(p_x\,v_x^3 + p_y\,v_y^3\right) \qquad (105)$$

It is also apparent that the total loss of power between Sections x and y is:

$$W\,F = \frac{W\,l}{2}\left(f_x + f_y\right) \qquad (106)$$

From Equations (105) and (106),

$$K\left(p_x\,v_x^3 + p_y\,v_y^3\right) = W\left(f_x + f_y\right) \qquad (107)$$

This reduces to

$$p_x\,v_x^3 + p_y\,v_y^3 = Q\,C^2\left(f_x + f_y\right)$$

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 120.

Equation (107) is the true power-loss equation rather than Equation (71), and hence the resulting Equation (76)* is, as Mr. Simham states, incomplete.

With reference to Mr. Simham's observations on the scouring of stream beds,† the velocity of the water in immediate contact with the wetted perimeter is the determining factor in scouring, rather than the mean velocity; so that one of the factors affecting scouring is the degree of velocity uniformity in the cross-section of the stream. This consideration would lead to the conclusion that the mean velocity required to produce scouring of a given material depends largely on the shape of the stream section and on local conditions and circumstances.

A consideration of the general principles governing weir discharge brings up many interesting digressions, one of which is discussed by Mr. Simham under the head of "Discharges from Catchment Basins".‡

The writer would call attention also to Equation (39),§ in which, H_0 is the elevation of a reservoir flow line above the crest of a spillway weir, Q is the coincident flow over the weir, and b is the net width of the stream on the weir crest.

Reservoir flow-line elevations are commonly computed from the Francis weir formula, $Q = c\, b\, H_0^{\frac{3}{2}}$, with the assumption that $c = 3.33$. This reduces to,

$$H_0 = 0.4484 \left(\frac{Q}{b}\right)^{\frac{2}{3}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (108)$$

The coefficient in Equation (39) is seen to be considerably greater than that in Equation (108). Furthermore, Equation (39) is predicated on zero friction losses in approaching the weir, and if appreciable friction losses exist the coefficient in Equation (39) would be even greater than 0.4717.

The trouble with Equation (108) is that its coefficient is obtained experimentally from measurements of H "a short distance up stream" from the weir, where the velocity of approach is assumed as being zero but where, as a matter of fact, there is undoubtedly some surface velocity even if $\frac{Q}{a}$ is negligibly small. The effects of these surface velocities are apparently not entirely eradicated even by the use of a stilling-box. Equation (38)‖ would indicate that the value of c in the Francis formula, corresponding to an absolute zero velocity of approach, is 3.087.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 120.

† *Loc. cit.*, p. 123.

‡ *Loc. cit.*, p. 125.

§ *Loc. cit.*, January, 1928, Papers and Discussions, p. 119.

‖ *Loc. cit.*, p. 119.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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THE O'SHAUGHNESSY DAM AND RESERVOIR

Discussion*

BY JOHN H. GREGORY, C. B. HOOVER, AND C. B. CORNELL, MEMBERS, AM. SOC. C. E.†

JOHN H. GREGORY,‡ C. B. HOOVER,§ AND C. B. CORNELL,‖ MEMBERS, AM. SOC. C. E. (by letter).¶—In his discussion,** Mr. Jakobsen mentions three points which merit further explanation, namely, the absence of any reference to the water-cement ratio, in connection with the mixing of the concrete; the methods of testing the concrete and the results of these tests; and the desirability of facing the lean concrete forming the main body of the dam with a richer mixture of concrete. As bearing on the first point, it should be stated again that concrete was first placed in the dam on November 1, 1922. At that time, engineers were generally of the opinion that there was a direct relation between the compressive strength of concrete and the quantity of water used in mixing. It had also been shown in laboratory experiments that the quantity of mixing water necessary to produce a workable concrete had a direct relation to the grading of the aggregates used, the fineness modulus being the means by which this relation was expressed. The application of these principles to large construction work and the development of satisfactory methods of field control were in the experimental stage. Little, if anything, was known concerning the weather-resisting properties of concrete proportioned according to the new theories, nor of its behavior when exposed to alternate wetting and drying, because long-time proof on these points was still lacking.

The engineers in charge of the design and construction of the O'Shaughnessy Dam were confronted with the problem of building a structure in which

* Discussion of the paper by John H. Gregory, C. B. Cornell, and C. B. Hoover, Members, Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Authors' closure.

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¶ Received by the Secretary, December 19, 1928.

** *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1327.

long life, a high degree of imperviousness, the greatest possible weight per unit of volume, and maximum resistance to the effects of temperature and water must be obtained; and, in which, the strength of the main body of the structure was of relatively minor importance, the maximum compressive stress being less than 100 lb. per sq. in.

At the time construction was started, it appeared to be very generally accepted by engineers that concrete proportioned to secure maximum density fulfilled these requirements, and, therefore, this basis of proportioning the concrete mixtures was adopted. In the prosecution of the work the quantity of mixing water was at all times held to the minimum amount which would secure the workability necessary for the satisfactory placing of the concrete, consideration, of course, being given to the location of the concrete being placed; and, furthermore, concrete was not transported by means of chutes because their use was not permitted by the specifications.

Numerous mechanical analyses of the four sizes of aggregates used were made and a large number of combinations of the results secured were plotted for comparison with the practical curve of maximum density, as established by Taylor and Thompson. Trial batches, containing cement, sand, coarse aggregates, and water, mixed by hand, were made up of the combinations which most nearly approached the Taylor and Thompson curve, and of variations from these combinations in which the total volume of aggregates was kept the same while the relative proportions of the several aggregates were changed. Each trial batch was measured for volume by placing the concrete in a pipe, the workability of the batch was observed and recorded, and, also, cylinders 6 in. in diameter and 12 in. long were made and broken, for a check on the compressive strength. From the results of these tests the various mixtures were selected. Throughout the work only slight variations from these proportions were made; for example, a small increase in the fine aggregate with a corresponding decrease in the coarse aggregates was allowed in thin walls and in the arch rings of the bridge; and, in the latter case, the cement, also, was slightly increased to secure greater workability.

From time to time throughout the course of the work test cylinders were made and broken. Cylinders made from concrete containing the two smaller sizes of coarse aggregate showed greater consistency in the test results than those containing the three sizes. It was demonstrated conclusively, in the case of a few of the tests, that cylinders 6 in. in diameter and 12 in. long were not as reliable as larger cylinders for testing concrete containing the 3-in. aggregate.

In addition to the proportioning and strength tests made by the engineers, J. R. Shank, Assoc. M. Am. Soc. C. E., was commissioned by the contractor to conduct an extensive series of tests, at the beginning of the work, with various combinations of the aggregates which were to be used. Several hundred test cylinders were made and broken and the results were substantially in accord with those secured by the engineers. Since none of the tests was made to set up a water-cement ratio for the work, a detailed account of the test results is thought to be out of place in this discussion.

A richer concrete facing (1:2:4), 1 ft. thick, was placed on the exposed faces of the dam and bridge piers for the purpose of securing greater resistance to scour, to alternate wetting and drying, and to changes of temperature. Experience with previous work in Columbus, especially the Julian Griggs Dam, convinced the writers that resistance to scour and to temperature and water exposure would be secured best and at a minimum expense by applying a 1:2:4 concrete on the face of the overflow and on the vertical sides of the abutment sections, as well as on other exposed concrete structures. With the facing used no difficulty has been experienced with shrinkage cracks in the Julian Griggs Dam and, to date (1929), none has appeared in the O'Shaughnessy Dam.

The discussion* of Messrs. Dittoe and Dixon raises a number of very interesting points. The most important of these has to do with scour at the toe of the two dams mentioned; that is, the Julian Griggs Dam, which was put under construction early in 1904 and practically completed in December, 1905, and the O'Shaughnessy Dam which was put under construction in 1922 and completed, ready for use, on June 1, 1925. Both structures are built on limestone, the geological formations at the two sites being, in general, similar.

Referring first to the Julian Griggs Dam, records have been kept by the City of Columbus since 1904, which show the scour at the toe of the dam. These records include elevations of the original surface of the ground and the top of the rock before construction started. Cross-sections were again taken on September 1, 1908, October 14, 1909, July 14, 1913, October 26, 1920, and the last ones on August 10, 1928.

Scour continued year after year, from the time the dam was completed in 1905 until 1920, that is, for about fifteen years, but, in the last eight years, since 1920, practically no further scour has occurred. On a few of the sections some further small scour has occurred since 1920, whereas, on the other hand, on some of the other sections there appears to have been a filling up. This is probably due to the movement of some loose rock on the bottom of the river. Taking the sections by and large it seems that a condition of practical stability below the toe of the dam was reached in 1920.

Attention should also be called to the fact that the dam, in March, 1913, passed through the most disastrous flood on record in the history of Central Ohio. At the peak of the flood on March 25, the depth of water passing over the 500-ft. spillway was 12.8 ft., with a flood discharge of approximately 80 000 cu. ft. per sec.

A full set of cross-sections was not taken in 1913, but scour occurred between 1909 and 1913. It is not known, however, whether the scour was progressive or whether it took place mainly at the time of the March, 1913, flood. The deepest scour, as of 1913, was at Stations 8 + 96 and 9 + 52 (see Fig. 26), since that time the river bottom has filled up considerably.

What is perhaps most important of all, careful soundings made in October, 1928, show that there has been no undercutting of the rock under the toe of the apron.

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1635.

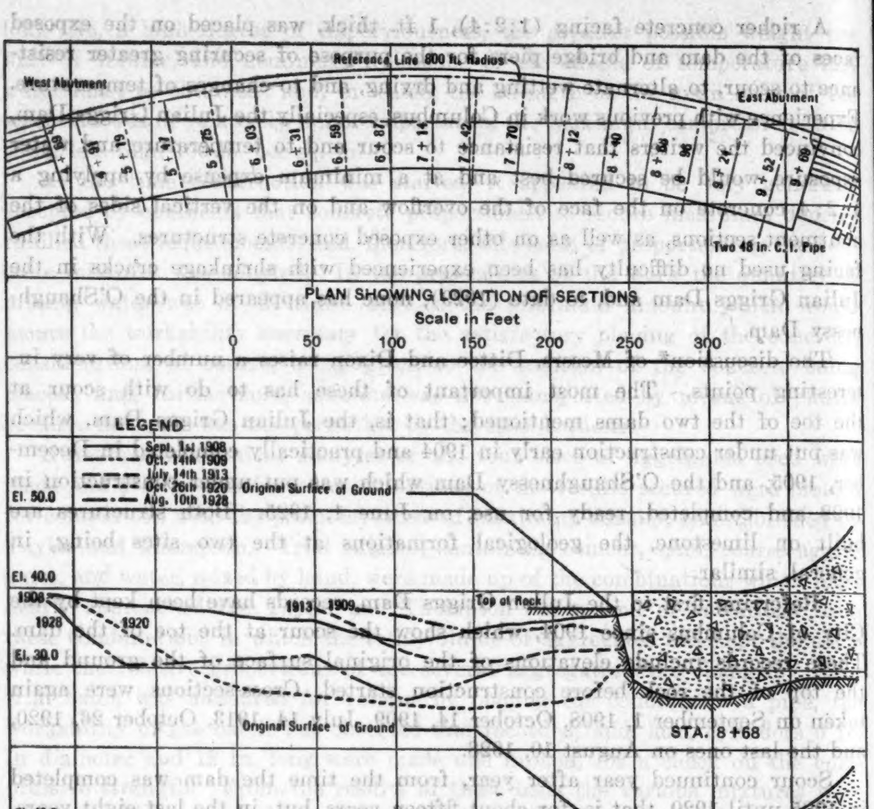

FIG. 26.—SCOUR AT TOE OF JULIAN GRIGGS DAM.

When the O'Shaughnessy Dam was designed the engineers had studied the records of the Julian Griggs Dam up to and including the 1920 soundings. As a result of these studies it was decided to build the O'Shaughnessy Dam, which is much higher, with a longer apron down stream, with a positive contact between the bed-rock and the concrete at the down-stream toe of the apron, and to excavate the rock down stream from the apron to an extent sufficient to provide a smooth discharge channel for the water, so as to prevent, if possible, scour such as had occurred at the Julian Griggs Dam. The extent of this lengthening may be obtained by comparing the cross-sections of the two dams.

On sections taken on the center lines of the first and second arches west of the outlet gate-house, some material left by the contractor after construction has been moved but little, if at all, by any high water so far experienced. On another section, taken on the center line of the third arch west of the outlet gate-house, there has been a little scour just down stream from the toe of the apron, but this is practically negligible (see Fig. 27).

As was the case with the Julian Griggs Dam, careful soundings made in November, 1928, show that there has been no undercutting of the rock under the toe of the apron, where the apron is below river level.

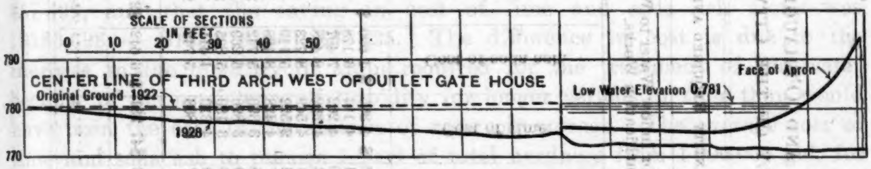

FIG. 27.—SCOUR AT TOE OF O'SHAUGHNESSY DAM.

There has been some scour in the spillway channels, above the river level, as was expected, especially at the edges where the water falls sidewise from one spillway channel to the one below. At the toe of the apron, at a few points, on different levels, there also has been some scour. It should be remembered, however, that the exposed rock in the spillway channels, in places, lies in rather thin layers, typical of limestone formations in and near Columbus, and it would be surprising if some scour had not occurred. Some of the scour was caused during high water in 1924 by cross-currents produced by forms and equipment stored by the contractor on the spillway channels; but such scour as has occurred has no bearing on the safety of the structure. These spillway channels are sometimes dry for periods of days and weeks at a time, and the voids in the rock can be very easily filled with concrete if the City should so desire, but there is no need to do so at present.

The writers have referred* to the apparent saving in the cost of water softening that resulted from reservoir storage during the period fom May 1 to September 10, 1926. This is supplemented in Table 8 by data that show the effect of storage in the Julian Griggs and O'Shaughnessy Reservoirs on the cost of purifying and softening the water supply of Columbus for a period of one full year from July 1, 1927, to July 1, 1928. Samples of water were collected daily from Mill Creek and the Upper Scioto, and at the end of each

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 468.

TABLE 8.—DATA SHOWING EFFECT OF THE JULIAN GRIGGS AND O'SHAUGHNESSY RESERVOIRS ON THE COST OF PURIFYING AND SOFTENING THE WATER SUPPLY OF THE CITY OF COLUMBUS, OHIO.

Month.	Consumption, in million gallons.	TOTAL HARDNESS IN PARTS PER MILLION.			Saving in cost due to decrease in hardness.	COST OF SOFTENING CHEMICALS PER TON.			QUANTITY REQUIRED AND COST OF COAGULATING AND SOFTENING CHEMICALS.													
									Actual, for Inflow Water to Water Purification Works.							Computed, for Inflow Water to O'Shaughnessy Reservoir.						
		At Water Purification Works.	At upper end of O'Shaughnessy Reservoir.	Decrease due to storage.		Lime.	Soda ash.	Alum.	Tons of lime.	Tons of soda ash.	Tons of alum.	Cost of lime.	Cost of soda ash.	Cost of alum.	Total cost.	Tons of lime.	Tons of soda ash.	Tons of alum.	Cost of lime.	Cost of soda ash.	Cost of alum.	Total cost.
July, 1927.	959	247	282	35	$ 2 694	$6.69	$28.60	$12.74	657	164	77	$ 4 395	$ 4 690	$ 981	$ 10 066	781	315	187	$ 5 225	$ 9 009	$ 1 745	$ 15 979
August, " .	915	244	338	94	6 903	6.69	28.60	12.74	615	216	74	4 114	6 178	943	11 235	884	442	65	5 914	12 641	828	19 383
September, " .	930	261	330	69	5 150	6.69	28.60	12.74	637	259	70	4 262	7 407	892	12 561	860	453	66	5 753	12 956	841	19 550
October, " .	890	280	352	72	5 165	6.69	28.60	12.74	646	294	74	4 322	8 408	943	13 673	920	460	64	6 155	13 156	815	20 126
November, " .	820	299	371	72	4 738	6.69	28.60	12.74	680	305	84	4 549	8 723	1 070	14 342	820	532	58	5 486	15 215	739	21 440
December, " .	865	148	282	134	9 303	6.69	28.60	12.74	377	179	314	2 522	5 119	4 000	11 641	638	338	124	4 268	9 667	1 580	15 515
January, 1928.	933	250	270	20	1 498	6.57	27.74	11.50	674	293	147	4 428	8 128	1 690	14 246	695	310	183	4 566	8 599	1 530	14 695
February, " .	830	192	246	54	3 597	6.57	27.74	11.50	516	207	268	3 390	5 742	3 082	12 214	556	292	178	3 653	8 100	2 047	13 800
March, " .	856	237	268	31	2 130	6.57	27.74	11.50	593	226	119	3 896	6 269	1 368	11 533	627	335	184	4 119	9 293	2 116	15 528
April, " .	816	189	255	66	4 822	6.57	27.74	11.50	484	210	299	3 180	5 825	3 338	12 343	575	247	117	3 778	6 852	1 345	11 975
May, " .	927	222	358	136	10 118	6.57	27.74	11.50	656	159	89	4 310	4 411	1 023	9 744	968	440	66	6 360	12 206	759	19 325
June, " .	878	206	273	67	4 721	6.57	27.74	11.50	588	207	227	3 863	5 742	2 610	12 215	696	330	188	4 573	9 154	2 162	15 889
Total.......	10 623	2 775	3 625	....	$60 839	.....	....	....	7 123	2 719	1 842	$47 231	$76 642	$21 940	$145 813	9 020	4 494	1 380	$59 850	$126 848	$16 507	$203 205
Average....	887	231	302	71	$ 5 028	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....

month composite samples of each were analyzed and the average analysis of the inflow water at the upper end of the O'Shaughnessy Reservoir was then taken as that secured by weighting the separate analyses on the basis of the areas of the contributing water-sheds.

The tons of lime, soda ash, and alum actually used at the Water Purification Works each month, together with the cost of each chemical and the total chemical cost, are given in Table 8. In this table also are given the computed tons of chemicals which the inflow water would have required, and the corresponding chemical cost if the inflow water had been treated so as to produce a water of the same quality as that delivered by the Water Purification Works.

The method of computation used in determining the tons of lime, soda ash, and alum required for the inflow water, when applied to the average monthly composition of the water actually treated, gives chemical tonnage values of about 4% less than those actually used in the treatment of the water, so that the method of computation, as applied to the inflow water, gives a conservative basis for comparison.

The tabulated data show that the annual saving in cost of the three chemicals required, due to reservoir storage, was (\$203 205 — \$145 813 =) \$57 392, and that the saving in cost of lime and soda ash alone was (\$186 698 — \$123 873 =) \$62 825. The difference in cost is due to the increase in the quantity of alum required for the treatment of the water because of the persistence of turbidity for longer periods of time than would have been the case in the absence of reservoir storage. The average cost of lime and soda ash to remove 1 part of total hardness from 1 000 000 gal. for the year was 8 cents. The saving in cost was, therefore, 10 623 000 000 gal. × 71 parts of total hardness × \$0.08 = \$60 339.

The total saving of \$57 392 in the cost of chemicals for the period of one year from July 1, 1927, to July 1, 1928, is equal to the annual interest on an investment of approximately \$1 150 000 at 5%; also, it represents a reduction of 28% in the cost of softening and purifying chemicals.

More than three years have now (1929) passed since the dam was completed, and a careful inspection of the structure fails to reveal any defects or any visible leakage around or through the dam.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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IMAGINATION IN CITY PLANNING

Discussion *

BY STEPHEN CHILD, M. AM. SOC. C. E.†

STEPHEN CHILD,‡ M. AM. SOC. C. E. (by letter).§—It is pleasant to note that the discussors were generally in agreement with the main argument of the paper. Every city, of course, is a living thing and needs planning until it ceases to grow; in fact, as has been recently called to the writer's attention, the history of city planning shows there is need for planning, or replanning, even after the period of rapid growth ceases. In other words, planning is a continuous process.

A method of procedure that should be useful in securing continuity in city planning, particularly for small and medium sized cities, has been stated previously by the writer.‖

Mr. Crosby is right in stating¶ that "too often the 'pictures' created have been of the 'still life' order"—beautifully illustrated reports soon laid aside and forgotten. The writer has quite a collection of such reports. Often the members of a city planning commission, when told that a previous report has been made, confess that they had never seen nor heard of it. Yet, in some instances, such reports have cost the community many thousands of dollars, nearly all of it wasted.

Mr. Crosby does well also to emphasize that "circulation", traffic by rail, motor, or on foot, must receive most careful study; that provision must be made for "peak loads" and, particularly, for future growth.

The inconvenience and economic loss due to the present-day "stop and go" control of traffic is becoming appalling and calls for imagination and for

* Discussion on the paper by Stephen Child, M. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Author's closure.

‡ Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

§ Received by the Secretary, December 26, 1928.

‖ *City Planning Quarterly*, October, 1927.

¶ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2147.

radical treatment. The writer has "tried his hand" at this in a previous paper.* Although somewhat radical, he believes that some such plan would be very helpful. It would have to be studied, of course, for each community.

The writer was pleased to note that Mr. Shurtleff† emphasized the importance of level-headedness along with imagination, and it was a particularly happy point to mention the need of co-operation in city planning on the part of engineers, landscape architects, architects, and sculptors. He might well have added city attorneys and other lawyers, housing officials, economists, and members of the other professions involved. The writer is glad to recall that, due to the thoughtful recommendation of Mr. F. L. Olmsted, a sincere effort was made in connection with the housing and town planning projects undertaken by the Government for war workers, and this sort of co-operation was used to the lasting benefit of the many projects that were planned.

The writer has particularly enjoyed the comments from Mr. Simham‡ and his approval of the need for civic orderliness and harmony—harmony is indeed "the fourth dimension" of city planning. It is most interesting to know that "India already possesses a comprehensive science of planning" that "follows the ancient spiritual ideals of the nation".

With all but one of Mr. Simham's thoughts, the writer is in heartiest accord. When, however, he recommends§ "that it [the commission] shall exercise the powers and perform the duties of the City Council in so far as city planning is concerned", and be not only an advisory but an executive body, it is necessary to state that, while in some cases this might be desirable—and the writer has often wished it were possible—such suggestions, unfortunately, run counter to the legal machinery in America.

* *American City Magazine*, April, 1927, p. 507.

† *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2149.

‡ *Loc. cit.*, October, 1928, Papers and Discussions, p. 2409.

§ *Loc. cit.*, p. 2412.

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PAPERS AND DISCUSSIONS

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LOAD DISTRIBUTION IN HIGH ARCH DAMS

Discussion*

BY MESSRS. FREDRIK VOGT, F. H. CONSTANT, AND ROBERT A. SUTHERLAND.†

FREDRIK VOGT,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—Under "Proposals for Improvements",‖ the author has emphasized varying the radius of curvature as the best way to make the arch better able to sustain the non-uniform load to which it is subjected. He refers to the Italian Government regulations which state that it is desirable to vary the curvature as may be required in order to maintain the thrust axial; but these regulations give no indication as to how this should be done.

In fact, it is quite impossible to solve the problem of maintaining the thrust axial and thereby to equalize the arch stresses by varying the curvature. The stress distribution may be improved in that way, but the stresses cannot be equalized as assumed in the Italian rules. Imagine that, for a certain shape, the thrust due to water pressure is axial at all points of the arch, and that no stresses existed before applying the load. With this axial thrust the arch would be compressed but not bent; the total length of the chord would thus be reduced. To make the arch fit the abutments a rib-shortening pull must be applied. This force is eccentric at all except two points of the arch and, therefore, it produces bending moments and bending stresses. It seems as if this well-known fact is overlooked sometimes.

In order to obtain axial thrust and uniform stress distribution under load, it is necessary to apply initial stresses in the arch. Heating the arch would do that in an ideal manner, but that is, of course, out of the question. Swelling of the concrete also would do it. However, the fact that the contraction

* Discussion on the paper by R. A. Sutherland, Assoc. M. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Author's closure.

‡ Cons. Engr., U. S. Bureau of Reclamation, Denver, Colo.

§ Received by the Secretary, October 15, 1928.

‖ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1043.

joints in dams can be grouted after some time shows that the shrinkage due to chemical action in the concrete and to cooling from the high setting temperature more than equalizes the swelling due to water-soaking; the final result is shrinkage even after the dam is entirely water-soaked. The cracks in many dams without joints show that the resulting shrinkage is usually greater than the extensibility of the concrete in tension. This condition means that the non-uniformity of the arch stresses is increased.

The question of how to equalize these arch stresses is of great importance since a thin arch with uniform stress distribution can carry the same load as a thicker arch with non-uniform distribution. The efficiency of arches will be highly increased if the stresses can be equalized and thereby a saving in material can be attained.

The writer can see no other way to obtain initial stresses for equalization of the final stresses than by use of a multiple-pressure grouting.* A division both vertically and horizontally of some of the contraction joints into compartments and the grouting of the different compartments under different pressures make it possible to obtain initial stresses distributed in about the same way as the stresses due to a heating of the dam. By means of such initial stresses in the arch not only can the arch stresses under load be approximately equalized, but the division of load between arches and cantilevers can also be checked in order to reduce the cantilever load as much as may be found convenient.

F. H. Constant,† M. Am. Soc. C. E. (by letter).‡—This paper is a continuation of the subject of the solution of arch dams by the trial load method, based on equal deflections of the horizontal arch and the vertical beam elements at every point of the dam. The method described was first suggested by William Cain, M. Am. Soc. C. E.§ It was later developed by F. A. Noetzli, M. Am. Soc. C. E.,‖ for the crown section only, under the assumption that the water load is uniformly distributed on each horizontal arch, and that the vertical beam is a cantilever fixed at the base. While the Noetzli method is also applicable to the case in which the vertical beam is hinged at the base, the additional condition introduced—that of zero moment at the base—greatly complicates the solution by trial. It may almost be said at the outset that the use of the trial method pre-supposes a dam with a fixed base. It can be expected, when the necessity for so doing is recognized, that efficient means for anchoring the dam to the bed-rock will be devised.

It was apparent immediately after Mr. Noetzli's paper was published, that there is no basis for the assumption of a uniform water load, and that the actual distribution cannot be determined by a study based merely on a coincidence of deflections at one section only. The paper¶ by C. H. Howell, M. Am.

* As outlined in his discussion of the paper entitled "Analysis of Arch Dams by the Trial Load Method," by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2515.

† Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

‡ Received by the Secretary, January 2, 1929.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2084.

‖ "Gravity and Arch Action in Curved Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

¶ "Analysis of Arch Dams by the Trial Load Method," *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 61.

Soc. C. E., and the late A. C. Jaquith, Esq., which met these defects squarely, and which demonstrated the practicability of the trial method in actual design, was "a long step forward". Mr. Sutherland's paper is a continuation of the same theme, and as, in the writer's opinion, the trial method, based on coincident deflections at every point of the dam, is the only practical solution of the problem, it is very welcome. Each contribution along this line helps to make clear some of the uncertainties and offers valuable suggestions to the designer. What is needed now is a definite outline of procedure, absolutely reliable formulas, reduced to their simplest terms and free from integral and other operating signs, and some preliminary idea of the water load distribution on the horizontal arches.

Each of the last two papers mentioned, gives valuable information in regard to water load distribution for actual dams; that by Mr. Sutherland, in particular, for the Salmon Creek Dam. It is interesting to note that he finds it is nearly uniform near the top (at Elevation 1 151), but that it becomes parabolic about one-third the way down (Elevation 1 103). Messrs. Howell and Jaquith also obtained a nearly uniform distribution on the top arch. The author states:* "It will be found that, for the lower arch sections, the load falls away rather rapidly from the center, whereas for the higher arch sections the loading varies less from a uniform distribution". This is valuable information for preliminary trials of future arch dams. More data of this kind are needed.

On account of the variation of the water load between the top and the lower arch sections, only formulas that will give influence deflection curves are of value. The author develops such formulas in Appendix II† and has translated them into curves (Fig. 15),‡ which, for the data on which the latter are based, are very valuable. Ultimately, it may be desirable to prepare a comprehensive set of such curves covering a wide variation of data, in order that the designer may be able to select the necessary influence lines from the curve without the labor of computing them from the formulas. The explanation of the use of curves is one of the valuable contributions of this paper.

The writer has previously given formulas for the horizontal thrust and bending moment at the crown and radial deflection at any point of an arch that is fixed at the ends and subjected to a pair of symmetrically placed radial unit loads.§ They were applied to the Stevenson Creek Dam to obtain influence values for deflections at each 10-ft. point for each 10-ft. horizontal section between the crest and the lowest point of the dam. A few experienced computers, using a calculating machine, and systematically tabulating results, can get all these values in two or three days.

The influence deflections thus obtained‖ were found to satisfy Maxwell's principle of reciprocal deflections fairly closely; that is, having two deflection

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1038.

† *Loc. cit.*, p. 1057.

‡ *Loc. cit.*, p. 1062.

§ *Loc. cit.*, May, 1928, Papers and Discussions, p. 1594; see, also, discussion by William Cain, M. Am. Soc. C. E., *Loc. cit.*, August, 1928, Papers and Discussions, p. 1894.

‖ *Loc. cit.*, p. 1595, Equation (61).

influence curves—for example, one for the 10-ft. point and the other for the 30-ft. point of an arch—the y-ordinate at the 30-ft. point of the first curve was equal to the y-ordinate at the 10-ft. point of the second curve.

The problem of the arch dam is still further complicated by the effects of temperature variation, shrinkage, etc., and much more study must be given to it before a final method of design is reached. It is just such contributions as the paper by Mr. Sutherland which gradually clear away these difficulties and lead to methods of shortening the work. For this, the author does indeed merit the thanks of the profession.

ROBERT A. SUTHERLAND,* ASSOC. M. AM. SOC. C. E. (by letter).†—Speaking of plastic flow, Professor Cain has stated‡ that "this is a consoling theory, but its realization was not much in evidence at the Stevenson Creek Experimental Dam * * *". The rate of loading in those experiments was, relatively speaking, instantaneous, and cracking was to be expected. In the "high" arch dams (more than, say, 150 ft.) with which this paper dealt, the rate of loading is generally slow, and plastic flow is a factor which must be considered. The writer's method of computing stresses under two extreme conditions corresponds with assuming: First, that the beams or cantilevers are cracked; and, second, that they are not cracked. By taking mean stresses a reasonable compromise is effected, which is the most that can be done at present. No theory can fit all the facts when dealing with such an imperfectly elastic material as concrete.

Professor Cain has shown previously§ that at one point on any assumed beam "all the water pressure is carried by the arch and none by the cantilever." The existence of such a point is, however, not incompatible with Figs. 5‖ or 18.¶ It should be emphasized that the "crest reaction" load shown in Figs. 4,** 5, and 18 is not an additional load on the dam, but simply part of the water load on the vertical beam transferred as a reaction to the crest support. To complete the load diagram, as far as the vertical beam is concerned, a horizontal arrow should be inserted at the base, directed up stream, to represent the remaining beam reaction and equalize the forces acting up stream and down stream. The effect of the crest reaction shown in Figs. 4, 5, and 18, is that, near the crest, certain loads due to part of the water load are forcing the beam down stream, while other loads due to the interaction of beam and arch are forcing the beam up stream. At one point, these forces will be equal, and the net force on the beam will be nil. This point will correspond with Professor Cain's point, K.††

Mr. Jakobsen‡‡ points out that the formulas given by the writer are correct only for relatively "thin" arches, such as $\frac{t}{r}$ less than 0.3. This does not

* Chf. Engr. and Asst. Mgr., Christmas Island Phosphate Co., Christmas Island.

† Received by the Secretary, December 19, 1928.

‡ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2151.

§ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1894.

‖ *Loc. cit.*, April, 1928, Papers and Discussions, p. 1037.

¶ *Loc. cit.*, p. 1068.

** *Loc. cit.*, p. 1035.

†† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 74, Fig. 15.

‡‡ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2154.

prevent their use for the majority of arch dams with relatively small error. It can easily be shown that the condition, $\frac{t}{r} < 0.3$, is very nearly equivalent to the condition that the depth is less than $\frac{3}{5} f$, in which, f is the cylinder stress, in pounds per square inch. Taking a usual value of 300 lb. per sq. in. (about 20 long tons per sq. ft.), this condition holds for plain arch dams up to 180 ft. high, and for higher constant-angle arch dams. For dams of greater height arch action is small in the lower part and errors made will be relatively unimportant even if $\frac{t}{r}$ is excessive.

Mr. Jakobsen has shown* that the writer's assumption of zero arch load at the abutments is incorrect. This is an important point in the theory of arch dams in so far as the resultant arch stresses at the abutments are concerned. It will not, however, have an appreciable effect on the deflections, on which the writer's method of load distribution depends, because the arch deflection ordinates fall away to zero so rapidly at the abutments. This will be clear from Fig. 5 (*d*), (*e*), (*j*), and (*k*), in which the arch deflection curves would be scarcely altered even if the arch load were given an appreciable value instead of zero at the abutment. The result on the arch stresses will be more important and will tend to increase the intradosal stresses at the abutment to more than those found by the writer.

The writer does not agree with Mr. Jakobsen that shrinkage has been neglected in the paper, since the manner of allowing for this was fully dealt with in the section entitled "Temperature and Shrinkage".† Since nearly all large arch dams are now built with vertical joints, shrinkage stresses are less important than in the case of a small dam like that of Stevenson Creek.

Mr. Bauman's interesting and constructive discussion emphasizes the utility of the ellipse of elasticity in the analysis of thrusts and deflections due to non-uniform and non-symmetrical loads.‡ He also shows that the cost of reinforcing steel required to take such tensile stresses as do occur in an arch dam is a very small percentage of the total cost. In spite of this the practice of reinforcing large arch dams has seldom been followed, probably on account of the difficulty of anchoring the bars adequately into the foundation, and of avoiding interference with the pouring of the concrete in bays. Most investigations on arch dams have aimed at devising methods of avoiding or reducing tension rather than accepting and providing for it, but the writer believes that the possibility of reinforcing arch dams might be profitably discussed.

Dr. Vogt states§ that the system of multiple-pressure grouting appears to be the only method of equalizing final arch stresses. This method would give ideal results for only one given set of conditions of water load and temperature, but it appears to hold out considerable promise in the direction of increased economy and security of arch dams.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2155.

† *Loc. cit.*, April, 1928, Papers and Discussions, p. 1039.

‡ *Loc. cit.*, October, 1928, Papers and Discussions, p. 2388.

§ See p. 804.

prevent their use for the majority of arch dams with relatively small error. It can easily be shown that the condition, $\frac{t}{r} < 0.3$, is very nearly equivalent to the condition that the depth is less than $2 f$, in which, f, is the cylinder stress, in pounds per square inch. Taking a usual value of 300 lb. per sq. in. (about 20 long tons per sq. ft.), this condition holds for plain arch dams up to 150 ft. high, and for higher constant-angle arch dams. For dams of greater height arch action is small in the lower part and errors made will be relatively unimportant even if $\frac{t}{r}$ is excessive.

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* Proceedings, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2155.

† Loc. cit., April, 1928, Papers and Discussions, p. 1030.

‡ Loc. cit., October, 1928, Papers and Discussions, p. 2388.

§ See p. 804.

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ADMINISTRATIVE WATER PROBLEMS
A SYMPOSIUM

Discussion*

By G. Clyde Baldwin, M. Am. Soc. C. E.†

G. Clyde Baldwin,‡ M. Am. Soc. C. E. (by letter).§—Discussion of the writer's paper has very properly not been limited to the problems incident to the transmission and delivery of water from Snake River reservoirs. Mr. Hinderlider‖ has presented an excellent summary of the methods used in the State of Colorado and has listed eight so-called requisites for equitable and efficient water administration which, with possibly slight modification, should be generally applicable.

The extent to which transmission losses are increased by "adverse-user" diversions, together with the accessibility of their head-gates, obviously will determine the degree of regulation which should be enforced. For example, it would probably be unwise to attempt to prevent the use of water by isolated canyon ranches unless the resulting benefits to be derived by the owners of older rights would be sufficient to justify the expense of the regulation.

Mr. Meeker¶ emphasizes the desirability of continued, comprehensive, engineering investigations and reports as an aid to better administration, and this is also stressed by Mr. Newell.** Both, however, recognize the variable, complex character of the different factors on which the stream flow at any given point depends, and they understand that exact solutions of river or reservoir loss problems can never be attained.

* Discussion on the Symposium on Administrative Water Problems continued from November, 1928, *Proceedings*.

† Author's closure.

‡ Hydr. Engr., U. S. Geological Survey, Idaho Falls, Idaho.

§ Received by the Secretary, January 22, 1929.

‖ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2179.

¶ *Loc. cit.*, p. 2181.

** *Loc. cit.*, November, 1928, Papers and Discussions, p. 2579.

Engineers usually are anxious to secure an increasing amount of data on such subjects in order to minimize the always recognizable error in the final application.

From the practical viewpoint of the ordinary water user, on the other hand, extreme accuracy seems less essential, especially if the engineering investigations must be done at his expense. Furthermore, in such extremely complicated problems as those on Snake River, the same basic facts may be used sometimes in good faith by different engineers to derive somewhat divergent conclusions. The water users, therefore, as a group, are rather slow to adopt radical changes in methods of operation and prefer to follow the usage of preceding years, with perhaps slight modifications based on new facts, rather than to sanction expenditures for engineering invesigations which may result in controversy.

As mentioned by Mr. Crandall,* segregation of stored water and normal stream flow at the outlets of reservoirs and the schedule of losses to be charged the former while in transit in the stream channel between the reservoirs and points of diversion are usually the two principal sources of contention in river administration.

The desirability of having these points settled by Court order or decree has long been recognized by those most interested in the storage and delivery of water on Snake River, and much has been accomplished through special studies and investigations with that end in view. At present (January, 1929), reports covering ground-water conditions along Snake River and the 1928 gains and losses in the American Falls Reservoir Basin are in process of preparation.

The net result of more than ten years of river operation has been the plan described in the paper which, as modified from time to time to meet new conditions or to be more consistent with added facts as they became available, has now, through precedent, attained a position almost comparable to, and may eventually be converted into, a Court order through stipulation by the interested parties.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2182.

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CHARACTERISTICS OF CENTRIFUGAL OIL PUMP

Discussion*

By Morrough P. O'Brien, Jun. Am. Soc. C. E.

Morrough P. O'Brien,† Jun. Am. Soc. C. E. (by letter).‡—The author's method of predicting the characteristics of an oil pump from its behavior when pumping water is somewhat similar to the method used in predicting the total resistance of a ship from tests of a geometrically similar model. The skin friction and the residual wave and eddy resistance are separated and stepped up to the full scale in different ratios, a procedure that is made necessary by the fact that the two resistances follow different laws of similarity. In the case of centrifugal pumps, the fluid is moving through closed conduits and hence no part of its resistance depends on the acceleration of gravity. For this reason, it should be possible to base the prediction of the performance with fluids of different viscosities on the Reynolds law of similarity which requires that the quantity, $\frac{v\,d}{\nu}$, be constant for dynamic similarity of flow.§

In this discussion, the notation is that used by the author, with the following necessary additions:

ρ = density of a liquid.
γ = weight per unit volume.
P = total power required.
p = power required to overcome bearing friction.

Considering two geometrically similar pumps of diameter, d and d_0, which are to be operated with fluids of kinematic viscosities, ν and ν_0, the condition for similarity is:

$$\frac{U\,d}{\nu} = \frac{U_0\,d_0}{\nu_0} \quad \text{.......................(25)}$$

* This discussion (of the paper by Michael D. Aisenstein, Esq., published in August, 1928, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof. of Mech. Eng., Univ. of California, Berkeley, Calif.

‡ Received by the Secretary, December 15, 1928.

§ "Die Grundlagen der Aehnlichkeits Mechanik and ihre Verwertung," Weber, Jahrbuch der Schiffbautechnischen Gesellschaft, 1919.

in which, U and U_0 are the velocities at the points to which d is measured. Substituting for the velocities in terms of the quantity flowing and the areas,

$$\frac{G\,d}{A\,\nu} = \frac{G_0\,d_0}{A_0\,\nu_0}$$

Since the two pumps are geometrically similar, the areas are proportional to the squares of the diameters and,

$$\frac{G}{G_0} = \frac{d\,\nu}{d_0\,\nu_0} \qquad (26)$$

To obtain the relation between the speeds of rotation, the velocity, U, is replaced in Equation (25) by the angular velocity, ω, and the diameter, d, with the result that:

$$\omega = \omega_0 \left(\frac{d_0}{d}\right)^2 \frac{\nu}{\nu_0} \qquad (27)$$

To determine the corresponding heads, the various components of the total head must be considered. For the pressure head, or head due to centrifugal action,

$$\frac{H'}{H_0'} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^2\,d_0^2\,\gamma}$$

in which, H' is the pressure head of the liquid, in feet; ρ is the density of the liquid; and γ is the weight per unit volume. Considering geometrically similar volumes, V and V_0, similarly placed in the two pumps, the kinetic energy is:

$$K\,E = V\,\rho\,\frac{\nu^2}{2}, \text{ and } K\,E_0 = V_0\,\rho_0\,\frac{\nu_0^2}{2} \qquad (28)$$

the velocity heads or kinetic energy per unit weight are,

$$H'' = \frac{V\,\rho\,\nu^2}{2\;V\,\gamma}, \text{ and } H_0'' = \frac{V_0\,\rho_0\,\nu_0^2}{2\;V_0\,\gamma_0} \qquad (29)$$

and the ratio of velocity heads is,

$$\frac{H''}{H_0''} = \frac{\rho\,\nu^2\,\gamma_0}{\rho_0\,\nu_0^2\,\gamma} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^2\,d_0^2\,\gamma} \qquad (30)$$

Losses due to impact are proportional to the velocity head, and will be in the same ratio.

The remaining element of the total head to be considered is the loss due to friction. Experiments on the flow of fluids in pipe lines have shown that for equal values of the Reynolds number, the values of the friction factor are identical. This may be expressed as:

$$\frac{R}{\rho\,\nu^2} = \frac{R_0}{\rho_0\,\nu_0^2} \qquad (31)$$

in which, R is the force per unit area resisting the motion. The resistance per unit area is proportional to the product of the loss of head, due to friction, H''', and the unit weight. Substituting this relation:

$$\frac{H'''\,\gamma}{H_0'''\,\gamma} = \frac{\rho\,\nu^2}{\rho_0\,\nu_0^2} \qquad (32)$$

or,

$$\frac{H'''}{H_0'''} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^{\,2}\,d_0^{\,2}\,\gamma} \qquad (33)$$

It appears that all the elements of the total head are in the same ratio for dynamically similar conditions.

To test this method, the rotational speeds of the two geometrically similar pumps should be in the relation given by Equation (27). The heads should then be adjusted, until the quantities are in the ratio given by Equation (26). If the Reynolds law of similarity is true, the observed heads should then be in the ratio,

$$\frac{H}{H_0} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^{\,2}\,d_0^{\,2}\,\gamma} \qquad (34)$$

The relation between the total power, P, required is,

$$\frac{P - p}{P_0 - p} = \frac{\nu^3\,\rho\,d_0}{\nu_0^{\,3}\,\rho_0\,d} \qquad (35)$$

in which, p is the power required to overcome bearing friction.

It should be noted that the geometric similarity of the two pumps should extend also to the surface roughness, which is a difficult condition to fulfill. For the purpose of checking this law of similarity, the impellers and casings might be given an artificial roughness proportional to the diameters. If the scale ratio is small, a rough agreement should be obtained by making both pumps of the same material.

If the scale ratio is unity, corresponding to the use of the same pump for both liquids, the speed ratio becomes,

$$\frac{\omega}{\omega_0} = \frac{\nu}{\nu_0} \qquad (36)$$

Should it be desirable to use the same speed of rotation, the diameters must be in the ratio,

$$\frac{d}{d_0} = \sqrt{\frac{\nu}{\nu_0}} \qquad (37)$$

The ratio of the kinematic viscosity of some common oils to that of water is about 18, making the fulfillment of Equation (36) difficult unless special testing equipment is used.

The author's method does not appear to be based on the condition of dynamic similarity of flow and, for this reason, it is applicable only in the range of types, diameters, and viscosities for which the coefficients have been determined. However, the method possesses the very definite advantage of simple experimental procedure. Unfortunately, the paper does not contain a comparison of the oil-pumping characteristics as predicted from water characteristics, and the actual oil-pumping characteristics as obtained by experiment.

$$\frac{H}{H_a} = \frac{\rho\,\omega^2\,d^2\,\gamma_a}{\rho_a\,\omega_a^2\,d_a^2\,\gamma} \qquad (33)$$

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$$\frac{H}{H_a} = \frac{\mu\,\omega^2\,d^2\,\gamma_a}{\mu_a\,\omega_a^2\,d_a^2\,\gamma} \qquad (34)$$

The relation between the total power, P, required is,

$$\frac{P-p}{P_a-p_a} = \frac{\nu^3\,\rho\,d}{\nu_a^3\,\rho_a\,d_a} \qquad (35)$$

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STREET DESIGNING FOR VARIOUS USES

Discussion*

By R. D. N. Simham, Assoc. M. Am. Soc. C. E.

R. D. N. Simham,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author states§ that the original purpose of a street was somewhat as given in dictionaries; that is, to serve as a public highway, paved or unpaved, in a city, town, or village. The writer considers that this definition does not imply that the function of a street was to serve traffic alone, because "way" may mean any one of a number of things according to the dictionary; but the only point that demands particular attention in the definition is that the words, town or city, in modern times means more than merely "a large collection of houses". The towns and cities of the Nineteenth and Twentieth Centuries may be spoken of as "almost exclusively industrial products", and therefore, they demand more facilities for intercommunication than similar cities in earlier times. To a town planner a "street" or highway is different from other "ways", such as railways, tramways, etc., only from the point of view of construction and utility.

The town and city planning experts of ancient India had very comprehensive ideas of streets. They established clear and definite rules for alignment, width, location, and direction; but, obviously, then they never had to take into consideration the heavy street traffic of modern times. Nevertheless, the combined effect of their wonderfully harmonious, systematic, and conscious planning has been to lessen and obviate almost all the present and most common difficulties with surprising success. They recognized the importance of light and air and recommended the orientation of streets north, south, east, and west. The old Hindu laws (the Smrithies) specified that the healthfulness of streets should be maintained by the proper circulation of air

* Discussion of the paper by George William Tillson, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Town Planning Asst., Madras, India.

‡ Received by the Secretary, January 8, 1929.

§ *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1725.

and by fair exposure to the rays of the sun and the moon. Streets were classified as: (*a*) Royal roads and streets; (*b*) main highways and crossways; (*c*) market or bazaar streets; (*d*) boulevards and circumscribing roads; (*e*) footways and footpaths; (*f*) back streets and lanes, etc.

According to one authority, the width of arteries and royal roads should be 30 "dhanus", or 180 ft., so that men, horses, elephants, cars, and vehicles could have free movement without interference and collision. Village highways should be 20 "dhanus", or 120 ft. wide. Main and circumscribing roads should be 60 to 120 ft. wide; streets and crossroads, 30 to 60 ft.; and village streets, back streets, and lanes, in towns, 24 to 30 ft. In a metropolis and in large cities, small streets and lanes were prohibited. Either the same thoroughfare might be allotted for different kinds of traffic or separate roads might be established. The greatest city was required to have seventeen thoroughfares in both directions; the average city, thirteen; and the smallest city, nine, which was the least number of thoroughfares permitted in any city worth its name. Towns could have from one to twelve such thoroughfares in each direction.

The author's reference* to some functions of a "street" does not include the entire range. The most important factors in the creation and growth of towns are: (1) Establishment of homes; (2) commerce; (3) manufacture and industries; (4) administrative forces; (5) physical attractiveness and healthful locations; (6) religious, social, and civic forces; and (7) education. Thus, street designing should be based on its relation to all these factors.

The writer considers an ideal street plan to consist of primary ways to serve as thoroughfares between cities and secondary ways to serve as streets for intercommunication between zones, parks, civic centers, etc. The primary streets may serve any of the needs of a city—local or otherwise—but the secondary streets are designed strictly for local use.

The effectiveness of a street system in a town plan is dependent on the "ideal" of the town. No imagination or vision in town planning will be of merit without such an "ideal". The dimensions and other details of streets must be based on the esthetic ideal of the community and the local needs of the city. The function of a street is to secure health, wealth, beauty, conveniences, sanitary conditions, and amenities of life for the city dwellers.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1725.

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THE RELATION BETWEEN EARTHQUAKES AND ENGINEERING SUBSTRUCTURES

Discussion*

By Fred A. Noetzli, M. Am. Soc. C. E.

Fred A. Noetzli,† M. Am. Soc. C. E. (by letter.)‡—This paper is of interest to the writer especially from the point of view of the effect of earthquakes on dams.

The author describes the behavior of a number of earth-fill dams and of one arched concrete gravity dam during earthquakes.§ In this connection, the writer wishes to mention the remarkable resistance of the Gibraltar Arch Dam, near Santa Barbara, Calif., and the Corfino Arch Dam, in Italy, which has been described by Alfred D. Flinn, M. Am. Soc. C. E.|| According to Mr. Stanley Bent, shortly after the completion of the Lake Hodges and the Murray multiple-arch dams, near San Diego, Calif., the surrounding country experienced a fairly severe earthquake. An immediate inspection failed to show any cracks or other visible signs of the effect of the earthquake on the structures.

Arranging dams in the order of greatest safety against earthquake the author places the earth-fill type first, the concrete gravity type second, and rock-fill, arch, and multiple-arch dams last.¶ There is hardly sufficient experience available to establish, with any degree of certainty, the relative merits of different types of dams in regard to earthquakes. Of interest in this connection are the author's statements that, "* * * a structure built on firm rock is not likely to suffer damage unless it is very weak or very near the fault of origin";** and that "the value of deep foundations for protection

* Discussion of the paper by Henry D. Dewell, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Cons. Hydr. Engr., Los Angeles, Calif.

‡ Received by the Secretary, December 15, 1928.

§ *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1741.

|| *Loc. cit.*, Part 3, Report on Arch Dam Investigation, May, 1928, p. 267.

¶ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1742.

** *Loc. cit.*, p. 1740.

against an earthquake seems quite definitely proved, * * *.* There is conclusive evidence that material 20 ft. or more below the surface of the ground does not move very much.

Thus, from the point of view of safety against earthquakes it is fortunate that most concrete dams are founded on rock, the surface of which is usually at least 20 ft. and sometimes 50 ft. and more below the original surface of the ground. It may be concluded, therefore, that from this point of view alone, properly built concrete dams of any type are not likely to be materially injured by earthquakes, unless they are constructed in the immediate vicinity of, or directly across, an earthquake fault.

Judging from the behavior, during earthquakes, of reinforced concrete structures of slender cross-sections, as compared with structures of plain concrete or masonry, it is the writer's opinion that reinforced concrete dams of the arch, multiple-arch, and Ambursen types should be structurally much less affected than the heavy and rigid type of gravity dam. An arch dam of 500 ft. span, for example, will deflect elastically about 1 in., due to stresses resulting from the water load. Under the influence of an earthquake wave, such a dam might be forced to deflect 2 or 3 in., without necessarily approaching, closely, the point of failure. Similarly, a lateral displacement of the side abutments of a multiple-arch dam, of 4 or 6 in., in a total span of 500 ft. (assuming, for instance, 10 arches at 50 ft. span each) would correspond to an average change of each span of 0.4 to 0.6 in., which certainly could not be considered fatal for reinforced concrete arches of the shape ordinarily used in multiple-arch dams.

On the other hand, the writer hesitates to predict what might happen if a rigid concrete gravity dam were subjected to the irresistible displacement of 4 to 6 in. in a 500 ft. span—either in shortening or in lengthening of the span.

The writer believes, therefore, that, contrary to the author's opinion, the slender and relatively flexible reinforced concrete type of dam—whether it is of the arch, multiple-arch, or deck-slab type—should be placed among the first rather than among the last in the list of the dams best suited to resist earthquakes.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1743.

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PUMPED-STORAGE HYDRO-ELECTRIC PLANTS

Discussion*

By Oren Reed, Assoc. M. Am. Soc. C. E.

Oren Reed,† Assoc. M. Am. Soc. C. E. (by letter).‡—With favorable topography and a low-head water supply the European expedient of pumping water into a storage reservoir for use at times of peak loads, as described by the author,§ may be very economical and may often have a decided advantage over a steam plant designed for similar service. The most costly demand on the electrical industry is to supply the power for the peak load, which lasts, on an average, only a few hours. In the last few years great strides have been made to meet this demand by providing storage hydro-electric plants as near the load centers as possible. This storage, in many cases, is maintained partly or entirely by pumping.

The rate of flow of some streams is similar to the power demand. This is not the case, however, in the central part of Europe, particularly in Northern Italy and Switzerland, and a proportionally greater value attaches to stored water. In these regions the head developed is usually very high, with a correspondingly small drainage area. The demand for power has greatly increased there in recent years and, on account of the high price of coal, the demand is being supplied largely by water-power plants. The streams are fed by melting snow and glaciers, and, therefore, they have their maximum flow in summer and minimum flow in winter. The present power demand, on the other hand, is greatest in winter, due to the lighting and heating load.

In the development of storage reservoirs, therefore, particular attention has been given to the possibility of supplementing the stream flow. Often plants of this type are operated only during the winter months while stream-flow plants of the same, or interconnected, system carry the summer load.

* Discussion on the paper by W. W. K. Freeman, Assoc. M. Am. Soc. C. S., continued from February, 1929, *Proceedings*.

† Asst. Designing Engr., San Joaquin Light & Power Corp., Fresno, Calif.

‡ Received by the Secretary, January 12, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2457.

As a result, it has been found to be economical to pump water from intermediate catchment basins into the primary reservoir in cases where storage is most valuable or where the release from storage is to be used through a high-head plant or a series of plants.

Because of the cheapness of brown coal in Germany, it is not economical to build hydro-electric plants, except for very favorable projects. A number of pumped-storage plants have been constructed to provide auxiliary power for the steam plants. The steam plant is run at a good load factor, and provides power for pumping at night and on Sundays. The stored water is then used during the periods of peak demand.

The efficiencies of motors, pumps, and hydraulic turbines have been brought to a point at which the losses in plants of this kind are not great. The over-all efficiency of the Herdecke pumped-storage plant in the Ruhr District, Germany, is between 60 and 65 per cent. The first plants of this type built in Germany had an over-all efficiency of less than 50 per cent.

Perhaps one of the greatest advantages of the hydro-electric stand-by would be the short time required to put the plant into service in an emergency. The plant could be started and synchronized with the system in a few minutes, whereas the best a steam plant could do would require several hours. Because of delay in heating boilers, the more important steam stand-bys are kept under steam at all times, and this necessitates a corresponding attendance and operating cost. Such cost might be eliminated entirely, and, in any case, would be less, if water power were substituted for steam in this service.

At some localities in Germany where the topography prevents the construction of pumped-storage hydro-electric plants, Diesel motor units are provided, as at Hamburg and Berlin. Such units have advantages similar to the pumped-storage plants, such as facility and quickness in starting in an emergency, and low charges when not in service.

In addition to the plants described by the author, the one at Tule River, California, and the Swiss plants at Arniberg, Palu, Cavaglia, and Mutten Lake deserve mention.

Tule River, California.—The Tule River project was completed in 1914. The flow of the Middle Fork of the river is diverted by a low concrete dam into a flow-line conduit of tunnels and ditches, leading to an open forebay above the power-house. The single impulse-wheel unit operates under a static head of 1 523 ft. About ½ mile below the diversion dam there is a permanent spring near the course of the stream. In 1922, a 5-in. centrifugal pump driven by a 100 h.p. direct-connected, electric motor was installed to pump the flow from the spring to the flow-line conduit on the hillside above. The pump operates automatically on a float-switch to deliver 2.2 sec-ft. against a head of 175 ft. through 1 100 ft. of 12-in. redwood pipe. A second unit, which was designed for a 200-ft. head, was installed in 1924. Each pump operates almost continuously from May to January of each year. The net increase at the power house is estimated to be three times the power input to the pumps.

*Arniberg, Switzerland.**—The Arniberg plant was completed in 1910. The flow of several small streams is diverted to a small forebay at the head of

* "Guide to Swiss Hydraulic Developments," 1926, p. 85.

the penstock at Elevation 4 503. The penstock crosses the Arni Brook at a point 307 ft. lower than the forebay pond. The discharge of this stream is pumped directly into the penstock. The power house operates under a static head of 2 795 ft.

Neither of the plants described operates entirely on the pumped-storage principle because pumping is not off-peak. However, a valuable block of firm power is obtained, which amply justifies the use of the pumping plant.

*Palu-Cavaglia, Switzerland.**—The two plants, Palu and Cavaglia, were completed by the Brusio Power Company in 1927. The upper plant, Palu, operates from the storage in Bernina Lake, which has a useful capacity of 12 150 acre-ft. The Cavaglia plant uses the flow from the Palu Glacier, in addition to the regulated flow. A short distance below Cavaglia is the intake to the Robbia plant of the same Company, completed in 1910.

The tunnel that supplies the Palu plant leads from the south dam at Bernina Lake; it has an internal area of 31.7 sq. ft. and a length of 3 570 ft. The penstock has an internal diameter of 43.2 in. and a length of 4 130 ft. The Palu plant operates under a static head of 1 013 ft. It contains a single vertical generator of 8 000 kw., driven by two turbines on the same shaft. The main turbine is near the generator level while the auxiliary wheel utilizes the additional head of about 85 ft. between the main turbine and the equalizing pond. A pumping plant is installed at Palu, designed for a maximum capacity of 21.2 sec-ft., to pump the summer flow from the Palu Glacier back into Bernina Lake. The flow from Bernina Lake is used in winter when the natural flow to Cavaglia and Robbia is small. The pumped storage is utilized in the three plants through a total static head of 3 703 ft. Below Robbia, there are two additional plants, Campocologno, with a head of 1 380 ft., and Poschiavino, with a 295-ft. head.

Mutten Lake, Switzerland (Proposed).†—Mutten Lake, which is at an elevation of 8 020 ft., has a drainage area of only 1.15 sq. mile. It is proposed to tap the lake by a 4 920-ft. tunnel to give a static head of 5 360 ft. at the proposed power house. The normal run-off from the lake is 5 070 acre-ft. per year, while the economical regulated capacity of the lake is 15 400 acre-ft. In order to supply the deficiency, it is planned to pump water from the Limmern River into Mutten Lake against a gross head of 2 425 ft. The pumping plant would have an installed capacity of 15 000 h.p. and would use, on an average, 25 650 000 kw-hr. of summer power, which would be generated by two stream-flow plants of the same system at off-peak periods.

* *Schweizerische Wasserwirtschaft*, June, 1926, p. 103, and August, 1928, p. 115.

† *Loc. cit.*, June, 1926, p. 105.

the penstock at Elevation 4 503. The penstock crosses the Arni Brook at a point 307 ft. lower than the forebay pond. The discharge of this stream is pumped directly into the penstock. The power house operates under a static head of 2 765 ft.

Neither of the plants described operates entirely on the pumped-storage principle because pumping is not off-peak. However, a valuable block of firm power is obtained, which amply justifies the use of the pumping plant.

Palü-Cavaglia, Switzerland.[3]—The two plants, Palü and Cavaglia, were completed by the Brusio Power Company in 1927. The upper plant, Palü, operates from the storage in Bernina Lake, which has a useful capacity of 12 150 acre-ft. The Cavaglia plant uses the flow from the Palü Glacier, in addition to the regulated flow. A short distance below Cavaglia is the intake to the Robbia plant of the same Company, completed in 1910.

The tunnel that supplies the Palü plant leads from the south dam at Bernina Lake; it has an internal area of 31.7 sq. ft. and a length of 3 570 ft. The penstock has an internal diameter of 43.3 in. and a length of 4 130 ft. The Palü plant operates under a static head of 1 010 ft. It contains a single vertical generator of 8 000 kw., driven by two turbines on the same shaft. The main turbine is near the generator level while the auxiliary wheel utilizes the additional head of about 35 ft. between the main turbine and the equalizing pond. A pumping plant is installed at Palü, designed for a maximum capacity of 21.2 sec-ft., to pump the summer flow from the Palü Glacier back into Bernina Lake. The flow from Bernina Lake is used in winter when the natural flow to Cavaglia and Robbia is small. The pumped storage is utilized in the three plants through a total static head of 3 703 ft. Below Robbia, there are two additional plants, Campocologno, with a head of 1 430 ft., and Poschiavino, with a 295-ft. head.

Mutten Lake, Switzerland (Proposed).[4]—Mutten Lake, which is at an elevation of 8 050 ft., has a drainage area of only 1.15 sq. mile. It is proposed to tap the lake by a 4 920-ft. tunnel to give a static head of 5 360 ft. at the proposed power house. The normal run-off from the lake is 5 070 acre-ft. per year, while the economical regulated capacity of the lake is 15 100 acre-ft. In order to supply the deficiency, it is planned to pump water from the Limmern River into Mutten Lake against a gross head of 2 435 ft. The pumping plant would have an installed capacity of 15 000 h.p. and would use, on an average, 25 050 000 kw-hr. of summer power, which would be generated by two stream-flow plants of the same system at off-peak periods.

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[3] *Schweizerische Wasserwirtschaft*, June, 1929, p. 106 and August, 1929, p. 119.

[4] *Loc. cit.*, June, 1929, p. 105.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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ECONOMIC COMPARISONS OF VARIOUS TYPES OF ROAD SURFACING

A SYMPOSIUM

Discussion*

By Messrs. William H. Connell, and Eugene L. Grant.

William H. Connell,† M. Am. Soc. C. E.—The speaker is very glad to note that the trend of the papers by Messrs. Everett and Kuelling are attempts to get down to fundamentals. He does not agree with all the conclusions in either paper, but both are on the "right track".

There has been too much talk of arriving at conclusions by considering only the original cost and the maintenance cost, and not considering interest and sinking-fund requirements. One thing that was omitted by both Mr. Everett and Mr. Kuelling is the sinking fund; that adds a little more to the cost of the higher types of road surfaces.

Both authors show that engineers are now (1929) studying their problems in a much more businesslike way than was the case a few years ago. The speaker believes that a great deal of this is due to the traffic surveys that have been made throughout the country, with a view to guiding the engineer with respect to what he should do, taking into consideration the prospective future traffic. That is the fundamental consideration.

Each author has pointed out that there is often a possibility of not having the funds to follow what is admittedly the correct procedure. Engineers will always be confronted with that difficulty. Many a man who builds a manufacturing plant finds that he cannot begin by doing what he would like. He has to "creep before he walks". Every mile of low-cost pavement that is constructed when there are not enough funds for anything better, and when it is absolutely essential and necessary to get the people out of the mud, will bring traffic to that stretch of the road. It will increase the sale and use of automobiles in the community through which that road passes. Eventually, a better type of road will have to be built, but the old road will have paid for itself in the meantime.

It always has occurred to the speaker that the pavement problem is one of "steps". This is not always the case. In certain communities the plan should

* Discussion of the Symposium on Economic Comparisons of Various Types of Road Surfacing continued from February, 1929, *Proceedings*.

† Executive Director, Tri-State Regional Planning Federation, Philadelphia, Pa.

be to begin by building the most desirable and best type; but in many outlying sections, in some States, and in some communities, where that is impossible and where it is nevertheless necessary to get the people to church and the children to school, it is most often advisable to provide a pavement that is only the first step.

Money spent in that first step will never be wasted, if drainage, alignment, grades, etc., are properly cared for, so that the second step may be installed later without wasting the first, and the third pavement, or step, may be built subsequently without wasting the second, etc. That is one of the principal considerations that every one should have in mind; and in comparing types of construction great care should be taken to consider them with respect to the traffic they are intended to serve. Mr. Everett has emphasized that very clearly.* He considers the light traffic, the medium, and the heavy traffic road, and endeavors to show that each of the paving types has its respective place under the conditions for which it is intended.

The speaker is very glad to note that more attention is being given to operating costs. Of course, highway transportation costs differ a little from railroad operating costs, because the latter pays the operating as well as the construction costs, while highway transportation operating costs are paid directly by the highway users. Railroad operating costs are paid from passenger fares and freight charges, but, after all, it is the same problem, because the public pays the bill.

Operating costs applied to motor vehicles are more or less new, and while a great deal has been done, engineers have just "scratched the surface". When the problem is considered from the same standpoint as that of the railroad, namely, that the fleet of cars or trucks that use the highways has its operating costs just as engines, freight cars, and passenger cars have, and that operating costs eventually must be paid by the public, then a true basis for comparing types of construction will have been found.

The principal fact that the speaker has gleaned from these papers is that engineers are now regarding their problems from a broad business as well as an engineering standpoint. While some of the operating costs may appear to be somewhat alarming, there is sufficient evidence to show that they are a far greater factor than the original cost of construction plus interest and sinking-fund and maintenance charges. If highway engineers will keep their minds focused on the whole problem, that is, the construction, maintenance, interest and sinking-fund, and operating costs, the speaker believes that, sooner or later, true values will be found for comparing the different types of construction under different traffic conditions.

Another factor is that roads should not be designed indiscriminately for a certain number of vehicles per day. A road constructed of a bituminous concrete pavement or some similar type of surfacing will stand up under a very large volume of passenger-car traffic. The same type of road between two large centers of population where there is not only a large passenger traffic, but also considerable truck traffic, would probably fail because of the truck traffic. A truck traffic equal to 10% of the total will do more damage than the remaining 90% of passenger traffic.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2478.

EUGENE L. GRANT,* ASSOC. M. AM. SOC. C. E.—The papers by Messrs. Everett and Kuelling are interesting not only for the information which they contain, but also because they illustrate different methods of approach to the problem of economic comparison. Highway engineers might well give consideration to selecting the best technique for presenting the results of an economic study. The selection of the best road surface for long-run economy is an example of the question that arises in all kinds of engineering activity, namely, "Can an extra additional investment now be justified by an anticipated future saving?"

There are three common ways of setting up this recurrent problem in engineering economy. In one method, the investments in alternative possibilities may be taken as giving rise to a certain interest charge and a certain depreciation charge, both of which may be considered as annual expenses. To these may be added the equated annual charges for maintenance and operating costs. The comparison of alternative plans may then be made on a basis which might be called "annual cost including interest".

In another method, all costs may be converted to an equivalent single figure as of some chosen date. This date may be at the beginning of the period of study, as in a "present worth" or "capitalized cost" comparison, or it may be at the end of the period of study as in the "compound amount" comparisons in Mr. Kuelling's paper.†

A third method, frequently used in commercial enterprises, but seldom in highway economy studies, would seem to have some advantages over both these plans; that is, express the annual saving effected by using the design with the lower operating cost instead of the one with the higher cost as a rate of return on the extra investment which it requires. In other words, determine the rate of return on an additional investment. If that return appears attractive as compared with other opportunities for investment and in light of conditions affecting the availability of funds, the higher cost installation is justified. For instance, if the extra cost of concrete over gravel is $20 000 per mile, and the annual savings are $500 in maintenance, $2 500 in tires, and $1 000 in gasoline, the saving is $4 000 per year, or a 20% return on an investment of $20 000.

Apply this method of analysis to the data in Tables 7‡ and 8.§ The extra investment in concrete is the difference between the $24 000 first cost of concrete and the $4 000 first cost of gravel, or $20 000. The equivalent annual savings from concrete are, as follows:

Maintenance	=	‖0.02868	×	$12 456	=	$357
Tires	=	0.02868	×	$88 236	=	2 531
Gas	=	0.02868	×	$27 304	=	783
Reconstruction	=	¶0.17918	×	$1 500	=	269
Total					=	$3 940
Less: **Depreciation	=	0.02868	×	$6 500	=	186
Net equivalent annual saving from concrete					=	$3 754

* Associate Prof., Civ. Eng., Montana State Coll., Bozeman, Mont.

† *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2482.

‡ *Loc. cit.*, p. 2487.

§ *Loc. cit.*, p. 2488.

‖ The annuity factor for 20 years and 5½ per cent.

¶ The annuity factor for 5 years and 5½ per cent.

** The total depreciation is $9 000 on the concrete and $2 500 on the gravel.

This represents a saving amounting to a return of 18.8% on the extra investment in concrete. Further analysis of the data indicates some interesting conclusions. For instance, if "indirect" costs for gasoline and tires are neglected, the saving to the State from reduced maintenance and reconstruction costs, less increased depreciation, will only be a 2.2% return on the extra investment in concrete. Thus, the only way in which the use of concrete may be justified over gravel in this situation is by a consideration of these "indirect" costs which are paid by the motor vehicle user and not by the State.

A study of the costs as shown year by year in Tables 7 and 8 indicates that the annual saving varies from a 9.9% return on the extra \$20 000 in the first year to 28.6% return in the twentieth year, resulting in an equivalent annual figure of 18.8 per cent. The immediate return rather than the equivalent annual return over the entire period of study might well be the determining factor in deciding how to spend available money where funds are limited and there were many projects on which expenditures would show a good return.

The expression of a saving as a rate of return on extra investment in an engineering economy study has the advantage of presenting the result in terms most intelligible to the average business man. In addition, it gets away, to some extent, from the mathematics of compound interest, which makes the problem seem complicated to the man who is not used to thinking in such terms.

It has also another advantage, which may be expressed as an answer to the question, "Is it justifiable to capitalize on the basis of 4½ or 5 per cent.?" The answer is implied when the highway engineer attempts it, and then finds that, while it is very economical to make a large investment, he cannot get the money.

What are the various opportunities for the spending of community funds either by the community or by individuals? If an extra investment in concrete over gravel will only net a 10% saving on the basis of all direct and indirect costs, perhaps there is some other field for community investment or for investment by individuals, which will net a greater saving.

Of course, in making practical application of the results of any cost study there are necessarily many judgment factors which cannot be expressed quantitatively, but which, nevertheless, may have a controlling effect on the final administrative decision as to which design should be selected. The arithmetical comparison of the economy of various designs is merely one of the aids to arriving at the proper administrative decision; but this limitation on the mathematical approach is just one additional reason for trying to make the results of the mathematical calculation tell the story in the best possible manner.

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RELATION OF THE LANDSCAPE ARCHITECT TO THE ALLIED PROFESSIONS ENGAGED IN CITY PLANNING

Discussion*

BY MESSRS. FREDERIC A. DELANO, STEPHEN CHILD, AND WILLIAM T. LYLE.

FREDERIC A. DELANO,† ESQ. (by letter).‡—The subject of city planning is so big that it ought to interest professional men who would approach it from totally different points of view; for example, it certainly is not a question solely for the architect or the landscape architect; no more is it a question solely for the engineer. The writer believes that lawyers and economists, such as those on the staff of the Russell Sage Foundation, and those interested from the viewpoint of social economics and community welfare, should take an active part.

The writer does not think that engineers have come down, ever yet, to the fundamentals of the question of zoning and taxation and their relation to the high building problem. For example, the authority which the Courts have given the administrators of cities to put limitations on the use which shall be made of property, is certainly based on certain fundamental propositions, namely, that private property shall not create an injury or a nuisance to adjacent property holders; that the city shall develop in an orderly way; and that an owner shall not make such use of his property that will deprive adjacent property of light, air, and enjoyment.

Beyond that, however, there is an economic side which has practically not been touched. Suppose, for the sake of illustration, that the City of Washington occupies an area of 5 000 city blocks. Assume that 25 city blocks at the most, with unrestricted height of buildings, were adequate to care for all the needs of the public in the way of office room and stores, and would earn in rents, interest, and taxes, on a valuation of $200 per sq. ft. or more of land

* Discussion of the paper by Arthur A. Shurtleff, Esq., continued from February, 1929, *Proceedings*.

† Chairman, Regional Plan of New York and Its Environs, New York, N. Y.

‡ Received by the Secretary, January 19, 1929.

area. Another 25 or 50 blocks might be devoted in the same way to hotels, theatres, and apartment buildings, and might earn interest and taxes on perhaps $25 to $50 per sq. ft. The remainder of the city might be used for minor business purposes, buildings of low elevation, or for private homes, but would probably be worth for those uses not to exceed from $1 to $5 per sq. ft. These figures are simply illustrative, but the point is this: It is to be doubted if a Court of Equity would rule that no commission could or should, under the guise of the orderly development of the city, say to one man that the use of his property should be limited so that it would be worth at the most only $5 per sq. ft., whereas another man should be so far privileged that his use of his property should yield him ten to twenty times as much.

The City of New York shows these conditions of terrifically high valuations on certain property, and near by there are the blighted conditions to which the writer has referred. This raises another question as to whether the city as a whole is taxed in order to furnish transportation facilities for the benefit of the few highly valued properties. In other words, is it the age-old story of the many being taxed for the benefit of the few?

STEPHEN CHILD,* M. AM. SOC. C. E. (by letter).†—It is good to have Mr. Shurtleff's statement‡ that "cities—whether old ones transformed for new uses, or new ones built for to-day—must give delight to the eye and to the spirit". It is also true, as the author states so forcefully, that this can only be accomplished by genuine co-operation on the part of the engineer, the landscape architect, the architect, and the several other groups mentioned.

The writer can testify to the important and helpful results attained by such co-operative effort in the work of the Government during the World War. As is well known, the Housing Corporation was formed within the Department of Labor to help solve the housing and town-planning problems of war workers, which arose from the acute congestion and over-crowding of that period. As a District Town Planner in the Town Planning Division of that Corporation, the writer took active part in the work of solving a number of such problems and is glad to bear testimony to the fine spirit of co-operation that prevailed, especially between the engineers, the architects, and the landscape architects, on the several problems on which he was engaged. There is no doubt that a large measure of the success attained by the Housing Corporation in the comparatively few months during which it functioned, was due to this whole-hearted spirit of co-operation.

Of the three professions mentioned, landscape architecture is, of course, the youngest, and, as Mr. Shurtleff states, it really began in this country with the work of Messrs. Frederick Law Olmsted and Calvert Vaux on Central Park, New York, N. Y. Never before, not even in Europe, had a public park been designed "from the ground up". Most of the great parks of Europe were originally hunting grounds of the nobility, in later years remodeled for public recreation. From ledges, bogs, and ash dumps of an area intrinsically ill-suited to its purpose, the designing power of the landscape architects developed

* Landscape Archt.; Consultant in City Planning, San Francisco, Calif.

† Received by the Secretary, January 2, 1929.

‡ *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2671.

Central Park. It is now (1929) more than 60 years that this area has contributed an overwhelming measure of genuine recreation to the people of Manhattan Island and vicinity.

It may be of interest to quote from Mr. Olmsted, an ideal of what such a public park should include:

"We want a ground to which people may go after their day's work is done, and where they may stroll for an hour seeing and feeling nothing of the bustle and jar of the streets * * *, the city put far from them. We want the greatest possible contrast with the town * * * consistent with convenience and the preservation of good order and neatness, * * * especially the greatest possible contrast with the restraining and confining conditions of the town * * *. What we most want is a simple, broad open space of clear greensward, with a sufficient play of surface and a sufficient number of trees about it to supply a variety of light and shade; these as a central feature. We want depth of wood enough about it, not only for comfort in hot weather, but to completely shut out the city from our landscape, * * *. The word, park, in town nomenclature should, I think, be reserved for grounds of the character and purpose thus described."

So successfully was this ideal accomplished at Central Park in New York, Prospect Park in Brooklyn, N. Y., Delaware Park in Buffalo, N. Y., Franklin Park in Boston, Mass., and elsewhere, that the demand for public parks spread rapidly all over the country. It soon carried along with it the demand for the design and construction of comprehensive systems of parks, each unit having a more or less distinct purpose. In the ideal system these are connected, by parkways, and beautiful avenues, both formal and informal in character, but all of ample proportions. It is well to emphasize, as Mr. Shurtleff does,* that "the relation between the city plan and these pleasure routes, and their relation to the sanitary improvement of river, stream, and pond borders within or near cities, was exceedingly intimate".

The writer is among those who believe that, if the problem of a comprehensive system of parks, including connecting parkways, for a city is really broadly solved, the planning of the city itself is practically completed; he believes that as often as practicable such comprehensive systems should be designed first. This, of course, cannot be done without the initiative and support, the heartiest co-operation of many groups of professional and business men, as mentioned in the paper. It is well to have this emphasized and re-iterated again and again.

Of the many successful examples of this sort of co-operation Mr. Shurtleff mentioned† Washington, D. C., as now being replanned. The writer would add, as earlier examples, Kansas City, Mo., Boston, Mass., and the outstanding example of Chicago, Ill. In the latter city, Mr. Daniel H. Burnham and the group of experts which his enthusiasm, tact, and skill, brought together, evolved what has correctly been termed the Burnham Plan, which is a design and a work of art (nothing less), but so scientific and so perfect in its underlying principles that it is to-day being executed bit by bit as special needs arise and funds permit.

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* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2669.

† *Loc. cit.*, p. 2671.

One very important unit has lately been completed—the Wacker Drive. This is indeed a beautiful structure, that cost $24 000 000 and is rightfully considered Chicago's show place. An important practical reason for this Drive is that it avoids congestion by running traffic around "the Loop" rather than through it. It is, of course, emphatically true that "the utilitarian must be at the base of every plan or it will fail"; but to the writer the most significant fact of all in regard to the great Wacker Drive improvement is that in practically its present form, with only the slight changes that time and especially automobile traffic conditions have made necessary, this feature was an integral part of the great Master Plan, the city planning design, developed under the guidance of Mr. Burnham in co-operation with engineers, architects, landscape architects, and other groups—a supreme example of their happy and whole-hearted co-operation.

WILLIAM T. LYLE,* M. AM. SOC. C. E. (by letter).†—This paper is devoted primarily to the subject of parks and parkways, which is pre-eminently a matter of landscape architecture, requiring the co-operation of the allied professions, with beauty as the chief desideratum. The history of the park movement is interesting in discovering the shifting emphasis from the small city park or square of a century ago (providing the site for a monument or two), to the admirable artistic creations of the Nineties, and more recently to modern parks and parkways in which a condition of maximum efficiency is sought by providing more fully for athletic recreation and the growing demand for improved transportation.

In city planning, also, the emphasis is shifting decidedly to the practical. The beautiful, however, will always play an important rôle. In the days of the World's Fair, at Chicago, Ill., the expression, "the city beautiful", was on every tongue. Since that time pressing problems of shipping, railroads, and street transportation have arisen, due to material prosperity and the increasing densities of population. The expression, "the city beautiful", is not in common use to-day (1929), although surely the city should be beautiful to be a fit place in which to live.

The important problems in most cities are those of transportation. These are first and foremost problems of engineering and have to do with such matters as harbors and ship channels, railroads, passenger and freight terminals, union stations, track elimination, and last, but not least, the traffic congestion in city streets. All these problems require the full and hearty co-operation of all the professions.

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

† Received by the Secretary, January 23, 1929.

MEMOIRS OF DECEASED MEMBERS

JOHN WESLEY BALL, M. Am. Soc. C. E.*

Died July 22, 1928.

John Wesley Ball was born at Walton, Ind., on August 8, 1888. He was the elder son of Joseph Ball and Sarah Ann (Johnson) Ball. He was graduated from the Galveston, Ind., High School in 1908, and from 1908 to 1910 attended the Indiana State Normal School. In 1910, he entered the Civil Engineering Department of Purdue University at Lafayette, Ind., from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1914; he received the degree of Civil Engineer in 1917. During the summer vacations of 1911 and 1912 he worked for the R. C. Bowman Company, of Cleveland, Ohio, and, during the 1913 vacation, as Engineering Assistant for the Cleveland, Cincinnati, Chicago and St. Louis Railway.

After his graduation in June, 1914, Mr. Ball, having passed the required Civil Service examination, was appointed by the late Logan Waller Page, M. Am. Soc. C. E., as Civil Engineering Student of the (then) Office of Public Roads in the United States Department of Agriculture. He worked in the Washington Office until July and was then made Chief of Survey Party on highway location in Sequoia and Yosemite National Parks, reporting to T. Warren Allen, M. Am. Soc. C. E., now Chief of Management in the U. S. Bureau of Public Roads.

A year of this work was succeeded by a transfer, in July, 1915, to Skamania County, Washington, and a promotion to the position of Locating and Designing Engineer for the Bureau on road construction under a \$210 000 bond issue in that County. Mr. Ball had sufficiently advanced this work by March, 1916, to permit his transfer to Jackson County, Oregon, to locate 70 miles of proposed highway between Medford, Ore. and Crater Lake National Park. This road survey was an outstanding job for excellence and low cost, and the present road follows closely the line run by Mr. Ball.

On the establishment in 1916 of a District Office at Portland, Ore., by the U. S. Bureau of Public Roads, Mr. Ball was attached to the Staff and during the season from March to November, 1917, he made investigations and reconnaissance surveys and prepared reports on National Forest Highway projects in Idaho, Washington, and Oregon.

On the entrance of the United States into the World War in April, 1917, Mr. Ball applied for admission to the Officers' Training Camp at American Lake, but was rejected on account of a defect in his heart action. In November of that year, however, he became Assistant Engineer for the U. S. Bureau of Public Roads on the paving work in that camp and remained there until February, 1918. During the spring and summer of 1918, he was at the Portland District Office. In October, he took charge of the construction

* Memoir prepared by L. I. Hewes, T. Warren Allen, and J. S. Bright, Members, Am. Soc. C. E.

of the Canyonville-Galesville National Forest Highway, in Oregon, which was completed in the fall of 1919. Mr. Ball then went to Lake Crescent, Washington, as Resident Engineer during the construction of the National Forest Highway there and remained until its completion in April, 1921. These two Forest Highways cost about $600 000 for 20 miles of grading.

When the U. S. Bureau of Public Roads established a Regional Office at San Francisco, Calif., Mr. Ball was selected to take administrative charge of National Forest highway construction in the eleven Western States and Alaska, reporting to L. I. Hewes, M. Am. Soc. C. E., Deputy Chief Engineer. This position he held with distinguished ability. In September, 1927, he was also placed in administrative charge of road construction by the Bureau in the National Parks in the Western States. He remained in charge of all this work in the National Forests and Parks until his death.

Mr. Ball was a man of unusual ability. In college, he was a member of both Phi Beta Kappa and Sigma Xi. His whole professional career was given to public service of a high order. He entered the U. S. Bureau of Public Roads in a minor position on graduation from college and through earnest and continuous application to his work rose step by step until he was filling the highest grade in the service. His technique was excellent and his love for perfect work outstanding. The development of modern standards of location and design in the National Forest and National Park roads owes much to the painstaking care of Mr. Ball in all details. His ability as an administrator constantly developed, and his work was characterized by a quality of humor and intelligence that drew people to him. He was a member of the Mystic Shrine and a Thirty-second Degree Mason.

In 1919, Mr. Ball was married to Ruth Stevenson, of Stevenson, Wash. He is survived by his widow and by a daughter, Elizabeth Ann, and a son, John Wesley Ball, Jr.

Mr. Ball was elected an Associate Member of the American Society of Civil Engineers on July 11, 1921, and a Member on March 15, 1926.

WILLIAM CHARLES BURKE, M. Am. Soc. C. E.*

Died July 21, 1928.

William Charles Burke was born near Montreal, Que., Canada, on February 1, 1857. He was the son of James and Mary (MacDonald) Burke. His father was of Irish ancestry and his mother of Scotch descent.

When Mr. Burke was fifteen years of age his family moved to Kansas City, Mo. where he completed his preliminary education. In 1874 and 1875, he attended Kansas University as a student of Civil Engineering, and during summer vacations, received his first practical engineering experience, acting as Assistant to the City Engineer of Kansas City.

From 1876 to 1880, he was employed by the Union Pacific Railway Company and rose rapidly from a subordinate position to that of Locating En-

* Memoir prepared by Morris C. Burke, Assoc. M. Am. Soc. C. E.

gineer. He made numerous location surveys for this Company in Western Kansas and Colorado during the period when Indian uprisings were not unusual, and military escorts were necessary.

During 1880 and 1881 he located and built the Missouri, Kansas, and Texas Railway from Fort Worth to Taylor, Tex., and, in 1882, was in charge of the erection of the Red River Bridge for the Cotton Belt Railway at Texarkana, Tex. This work was carried on under serious difficulties, as malarial fever was very prevalent and Mr. Burke was forced, on the completion of the railroad, to seek a more healthful climate. He went to San Marcial, N. Mex., as Division Engineer in charge of maintenance of way, of the Rio Grande Division of the Atchison, Topeka and Santa Fé Railway, in which position he remained until 1884 when he was appointed Assistant Chief Engineer of the Kansas City, Clinton, and Springfield Railway. He located and built this line from Kansas City to Springfield, Mo.

He spent the next two years, 1885 and 1886, with the Union Pacific Railway Company, locating a line from Salina to Lincoln, Kans., and relocating and constructing the line from Denver to Boulder, Colo. On the completion of this work, in 1887, Mr. Burke returned to the Atchison, Topeka and Santa Fé Railway Company, and located and built its main line from Fort Worth to Purcell, Okla. After this work was finished, he located and built the line of the Missouri Pacific Railway from Marquette to Gypsum City, Kans.

From 1888 to 1890, he was associated with two of his brothers-in-law under the firm name of Colt, Reinhart, and Burke, Contractors. They constructed the Missouri Pacific Railway from McCracken, Kans., to Pueblo, Colo., and the Fort Lyon Irrigation Canal in the Arkansas Valley, Colorado. The firm promoted, designed, and built the Rocky Ford, Colorado, Highline Canal, and numerous small projects, and was also engaged in the mercantile business, milling, and sheep raising. During the period from 1883 to 1890 Mr. Burke was also associated with his brother, Thomas Burke, in the cattle business, their ranch headquarters being thirty miles northwest of Prescott, Ariz.

From 1890 to 1897, Mr. Burke lived at Las Animas, Colo., engaging in various activities. He made a report to the U. S. Department of the Interior as to irrigation possibilities in the Ute Indian Reservation. He was Consulting Engineer for the Great Plains Water Storage Company of Holly, Colo., and served for years as Receiver of the Fort Lyon Canal Company. He also made various reports of railroad and irrigation projects in the West and in Mexico.

In 1898 and 1899, as a member of the contracting firm of J. B. Colt and Sons Company, he built the St. Louis and North Arkansas Railway from Eureka Springs to Harrison, Ark. In 1901, Mr. Burke located the Eastern Oklahoma Branch of the Sante Fé Railway from Newkirk to Pauls Valley, Okla., and revised the location of the main line from Arkansas City, Kans., to Newkirk, Okla. During 1902 and 1903, as Division Engineer, he constructed the Missouri, Kansas, and Texas Railroad from Coffeyville, Kans., to Oklahoma City.

In 1904 Mr. Burke was appointed City Engineer of Oklahoma City, Okla., and remained in this position until 1911. During these years, the city grew very

rapidly and the problem of keeping the improvements apace with the growth was tremendous.

From 1911 to the time of his death, he lived and maintained his office in Oklahoma City, and his consulting practice covered numerous projects of all descriptions, the more important of which were, as follows: Chief Engineer, Municipal Engineering and Construction Company; Consulting Engineer, State of Oklahoma, in the Two-Cent Rate Case; Chief Engineer, Cameron County Water Improvement District No. 1, Harlingen, Tex.; Chief Engineer, Lee Land Company, Lower Rio Grande Valley, Texas; Chief Engineer, Rancho de Santa Maria, Texas; Chief Engineer, Municipal Excavator Company, Oklahoma City; Chief Engineer, State Conservation Commission of Oklahoma; Chief Engineer, North Canadian River Flood Control District No. 1, Oklahoma City; Consulting Engineer, Oklahoma Natural Gas Company; Consulting Engineer, Atchison, Topeka and Santa Fé Railway; Consulting Engineer, Citizens Committee of Oklahoma City on Union Railway Station controversy; and senior member of the firm of Burke Engineering Company.

Mr. Burke was married in 1887 to Daisy May Colt, of Clinton, Mo. He is survived by his widow, two sons, Morris Colt Burke, Assoc. M. Am. Soc. C. E., of Oklahoma City, and James MacDonald Burke, of La Jolla, Calif., and two daughters, Mary Francis Burke and Harriett Elizabeth Burke.

Mr. Burke's life was a splendid example of devotion to duty, to his profession, to his employers, and to his family. He was admired and loved by all who knew him and numbered his friends by the thousands throughout the Southwest. The history of his life is practically the history of the building of the West and a personal narrative of his career would be rich with romance and adventure.

Mr. Burke was elected a Member of the American Society of Civil Engineers on April 12, 1926.

SILAS MAXWELL HAIGHT, M. Am. Soc. C. E.*

Died September 29, 1928.

Silas Maxwell Haight was born in Elmira, N. Y., on May 5, 1883. His father, Maxwell Haight, one of those high-minded, great-hearted, jovial men who endear themselves to all, enjoyed a wide popularity in his community. His mother was a woman of beautiful character and great courage whose maiden name was Frances Hendy. She was a direct descendant of John Hendy who held a commission as Colonel in General Washington's Army during the Revolutionary War, himself a surveyor and engineer. He was the first white settler in the Chemung Valley and his descendants have always lived in or near the present City of Elmira.

* Memoir prepared by M. W. Wipfler, M. Am. Soc. C. E.

Mr. Haight's early education was acquired in the public schools and High School in Elmira. He afterward entered Lafayette College, at Easton, Pa., from which he was graduated in June, 1908, with the degree of Civil Engineer.

His first professional engagement was with the City of Elmira as Transitman in the Engineering Department, where he soon assumed responsible charge of much municipal work. He remained with the City of Elmira as Construction Engineer until the outbreak of the World War, and in August, 1917, he entered the Officers Training Camp, at Fort Niagara, New York, where he was commissioned a First Lieutenant. He was assigned to duty overseas and attached to the 475th Aero Squadron, American Expeditionary Force. After service in France, he was ordered to Lopcombe Corner, England, where he was placed in full charge of the design and construction of airdrome buildings and pertinent works.

On his return after the war, Mr. Haight resigned his municipal position to take up private engineering work in which he was very successful. He constructed many miles of brick and concrete pavements, and was Resident Engineer during the building of the beautiful reinforced concrete bridge in Elmira which spans the Chemung River at Main Street. After many other private engineering accomplishments, Mr. Haight accepted an important position as Engineer with the Highway Products and Manufacturing Company, manufacturers of Armco corrugated iron pipe. He assumed complete charge of the design and installation on many large drainage projects, flumes, sewers, etc., and his work in iron-pipe promotion was so pronounced and successful that he soon attracted the attention of several other manufacturers who accordingly bid for his services. Just a few months before his death, he had accepted a responsible position with the Sweet Steel Company, of Williamsport, Pa., and he was taking a business trip for this Company when he was instantly killed in a motor car accident, in the prime of his busy life.

"Si" Haight, as he was familiarly called, had friends by the thousands. He was sociable and kind, with a great mind and memory. Blessed as he was with a keen sense of humor and happy disposition, he won warm friends wherever he traveled. He was a gentleman at all times, big-hearted, noble, and brave, and ever loyal to his legion of friends who mourn his loss.

Immediately following Mr. Haight's funeral, which was largely attended, several of his more intimate friends met at the City Club in Elmira and perfected a permanent organization known as the "Si Haight Club". The members of this Club intend to meet once each year on the anniversary of Mr. Haight's death and hold a banquet in his honor, at which time fitting tributes will be extended to his memory.

Mr. Haight was a member of Union Lodge No. 95, F. and A. M., Elmira; the Corning Consistory; Kalurah Temple, A. A. O. N. M. S.; the Southern Tier Shrine Association; Elmira Lodge of Elks No. 62; Phi Delta Theta Fraternity, and various other organizations.

He was unmarried and was always devoted to his mother and to his sister, Mrs. Grace Haight Parker, who is his sole survivor.

Mr. Haight was elected a Member of the American Society of Civil Engineers on June 6, 1927.

GUSTAV LEHLBACH, M. Am. Soc. C. E.*

Died March 26, 1926.

Gustav Lehlbach was born in Baden, Germany, on July 3, 1844. The son of a Lutheran minister of progressive tendencies—a man who thought of principle first and safety afterward—he was brought to this country by his parents when, with Carl Schurz and Baron Friedrich von Steuben, they were forced to flee from Germany on account of his father's active participation in the rebellion of 1848. The family settled in Newark, N. J., where the Rev. Mr. Lehlbach accepted the pastorate of the old Mulberry Street German Evangelical Church.

Gustav Lehlbach attended the public schools in Newark and then started his preparation for an engineering career in the old way—by apprenticeship to a successful engineer. This connection was severed temporarily when, at the age of sixteen, at the outbreak of the Civil War, he marched off as a volunteer in General Kearney's unit. The pioneering spirit of his forebears found early expression when, one cold March morning, Mr. Lehlbach, with a single comrade, swam the Rappahannock River stripped to the skin, in order to be the first in the enemy's rifle-pits. Fortunately, the two adventurers were right in their conjecture that the Confederates had abandoned the pits during the night.

In 1861 he was severely wounded in his first engagement at the Battle of South Mountain. On his recovery he was assigned to the Signal Corps, and, toward the close of the war, was stationed as one of President Lincoln's bodyguard at Arlington Headquarters. Whether from the contacts of that period, or from subsequently acquired appreciations, he, like so many others on both sides of the Mason and Dixon Line, came to have an admiration for Abraham Lincoln which grew to reverence deep enough perhaps to have had something to do with later attitudes and ideals. Particularly in his later judgments of men in public life, and in the formation of his own views on political, social, and religious subjects, Mr. Lehlbach was much influenced by Lincoln—and by Jesus of Nazareth. He saw the Nazarene as Lincoln saw Him—clearly, undistorted by the refracting media of church, sect, and dogma.

After General Lee's surrender at Appomattox, Mr. Lehlbach took up his engineering apprenticeship where he had left it in 1861. In two short years he was Resident Engineer, locating and constructing the Newark and New York Railroad, not then a branch of the New Jersey Central, but an independent project backed by Newark men. Owing to his foresight, no grade crossings over Newark's streets were permitted, except at Mulberry Street, and this was eliminated later. He also sought to have the line carried at a high level over Broad Street to tap the Oranges, but in this he was overruled.

Bridge design in those days was a very much simpler matter than it is now; so that by continuous hard work, made possible by an exceptionally sturdy physique, Mr. Lehlbach acquired a working knowledge of that subject,

* Memoir prepared by Henry G. Babcock, Assoc. M. Am. Soc. C. E.

and, if the writer's recollection serves him rightly, was thus enabled to design the bridge across the Passaic River. The caisson foundations for the center pier of this old swing-bridge were, it is believed, his choice of construction method and were sunk under his direction.

From 1869 until the fall of 1871 he was Chief Engineer on the location and construction of the Port Royal Railroad from Augusta, Ga., to Port Royal, S. C., and subsequently Resident Engineer on the construction of the New York and Long Branch Railroad. On April 19, 1872, he became, as City Surveyor of Newark, the head of the Engineering Department of that city.

For five years, from 1875 to 1880, Mr. Lehlbach was engaged in private practice as a partner in the firm of Lehlbach Brothers at 196 Market Street, Newark, a business still carried on at the same address by his nephews, Herman B. and Milton Lehlbach.

In 1880, came another chance for that railroad pioneering which enlisted his enthusiasm more than any other engineering activity. First, in this period, came the Dover and Rockaway Railroad for the Central Railroad Company of New Jersey. Then, about 1883, he located the line of the Denver and Rio Grande Western Railroad, in Utah. Accurate information covering this part of Mr. Lehlbach's life is meager, but probably there should be assigned to it an episode in itself well enough authenticated. While engaged in reconnaissance, Mr. Lehlbach covered many miles on horseback and unaccompanied. Many of these lonely expeditions were through uninhabited desert, and on one of them he unwittingly drank some of the poisonous alkaline water in which those deserts abound. Soon taken violently ill and unable to ride farther, he dismounted and lay helpless on the sand through two or three days, each of which must have seemed weeks, of the burning desert heat, while his horse foraged at will, presumably with very little success. The attack finally wore off. He recovered his horse and rode back into camp.

Similar episodes attended his work in Mexico where he acted for the Mexican Government, once more on railroad reconnaissance. Although his party numbered 30 or 40 men, Mr. Lehlbach, as usual, made many solitary expeditions. On one of these occasions, in attempting to ford a stream which he had traced for 30 miles, his horse suddenly struck quicksand. By grasping the overhanging limbs of a tree, he was able to pull himself to safety and then to coax and encourage his horse to struggle back to *terra firma*. Unable, on account of the quicksand, to retrace his steps, he seemed to be facing another desperate situation, as he knew of no settlement within a hundred miles. He gave his horse the bridle and the creature's instinct brought him to a trapper's cabin where he found food and shelter for the night, and whence he eventually found his way back to camp.

While in Mexico Mr. Lehlbach experienced real Mexican hospitality in the home of General Bustamante. During the Civil War the General had been the delegate in Washington of the Mexican patriots who were trying to oust Maximilian, and were seeking President Lincoln's help. President Lincoln, as is well known, could not see his way clear to give material assistance or even recognition and open moral support to the enemies of Maximilian. Yet, he

evidently left no doubt in Bustamante's mind as to where his personal sympathies lay—and he must have left an indelible impression of his personality on the General. Perhaps, when Mr. Lehlbach asked General Bustamante to describe that impression, he was, in memory, mounting guard again at Arlington. Certain it is, however, that the Mexican General's reply found many an audience in Mr. Lehlbach's later days when railroad reconnaissance had become reminiscence. Said Bustamante, "He was another Jesus Christ."

Mr. Lehlbach was married to Anna Lydia Babcock, of Communipaw, N. J., on January 17, 1872. During his married life his engineering business came to take the place of railroad pioneering, and he eventually settled down in Newark where he ended his days. Toward the close, some visitors to his Newark home were perhaps curious about a sort of double-decked picture frame which hung from the library mantle. The lower story enclosed a portrait of President Woodrow Wilson for whom Mr. Lehlbach had acquired a warm admiration; but from the upper story of the frame, above the War President of 1917, looked out the face of the War President of 1861. The visitor who remarked about that double-decked portrait was quickly made to feel how much it meant to Mr. Lehlbach. It evidently had to do with the boyhood days at Arlington, with dreams and ideals revivified for him by the words and acts of the original of the lower portrait.

He died on March 26, 1926, his widow surviving him by less than two years. He had no children.

Mr. Lehlbach was elected a Member of the American Society of Civil Engineers on March 7, 1883.

EUGENE AUGUSTUS HOFFMAN TAYS, M. Am. Soc. C. E.*

Died July 22, 1928.

Eugene Augustus Hoffman Tays was born at West Point, N. Y., on October 24, 1861, the son of the Rev. Joseph Wilkin Tays and Jennie (Crowell) Tays. At the time of his birth his father was teaching mathematics at the United States Military Academy. His ancestry on his father's side was Scotch-Irish, whereas on his mother's side it was English.

Mr. Tays received his early training in the Maysville, Ky., Institute and the Burlington, N. J., Military College. In the fall of 1880 he matriculated at Union College, Schenectady, N. Y., where he studied Civil Engineering, but owing to illness he was compelled to leave college at the close of his second year. While he was at Union he joined the Delta Upsilon Fraternity, and took several prizes in his military course for tactics and marksmanship.

Mr. Tays' first work in his chosen field was as Rodman, Leveler, and Topographer for the Mexican Central Railroad Company, on which road he later served as Division Engineer of Construction (1882-1883). In 1884 he was an

* Memoir prepared by the Rev. W. N. P. Dailey, New York, N. Y.

Engineer on the Topolobampo and Pacific Railroad, and in 1885 he was engaged as Engineer in charge of the irrigation of a vast tract of land in the District of Fuerte, Sinaloa, Mexico.

In 1886, Mr. Tays went into private practice and was employed successively by the Mexican-American Construction Company of Sinaloa, the Sinaloa and Chihuahua Railroad Company, and the Mexican Western Railroad Company. In 1891 and 1892, he was Chief Engineer of the Topolobampo Canal, and, in 1893, Engineer and Superintendent of the San Javier Silver Mines of Sinaloa.

At this time Mr. Tays left the field of Civil Engineering for that of Mining Engineering, in which he became an expert, called on by the United States Government in expert testimony. He spent twenty or more years in this field of endeavor. During this time he was connected with the Palmarejo Mining Company of Palmarejo, Chihuahua, Mexico; the Mexican Mineral Railroad Company; the San Agustin Mining Company of San Rafael, Chihuahua; the Anglo-Mexican Mining Company of San José de Gracia, Sinaloa; the Companía Minera de Santo Tomás y Anexas at San José de Gracia; the Maconí Mines, Maconí, Querétaro, Mexico; the United Mining Company of New York; the United Sugar Companies, Los Mochis, Sinaloa, Mexico; and the Companía Minera Occidental Mexicana of South America, at San Blas, Mexico.

Because of the unsettled conditions in mining operations in Mexico in 1924, Mr. Tays turned to agriculture, and organized the Liga Agrícola Occidental Mexicana, of which he was the Managing Director. He was engaged in this most successful venture at the time of his death.

Mr. Tays was a constant contributor to scientific publications on mining, milling, metallurgy, geology, etc. One of his contributions was on "The Economics of Mining". His studies as published in the *Mining World, Engineering and Mining Journal, Mining and Scientific Press, Western Mining World,* and the *Transactions* of the American Institute of Mining and Metallurgical Engineers, are notable contributions. He was affiliated with the following technical, scientific, and social organizations: The American Institute of Mining and Metallurgical Engineers; the United States Public Reserve (during the World War); the American Geographical Society; the Santa Cruz Club, of Nogales, Ariz.; the Society of Arts, London, England, and the American Association for the Advancement of Science.

He was married, in 1885, to Rosaura Vega, daughter of the Governor of the District of Fuerte, Sinaloa, Mexico. They had eight children, José W., Eloisa, Eugene, Clement I, Linda, George, Alexander, and Clement II, six of whom, with his widow, survive him.

Mr. Tays seems to have had at least three strong ties with which he bound himself up in life—his home and family, his profession and service, and his college and class. An Episcopalian in religion, he nevertheless held himself as a servant of all men wherever he might be of use, and through the years of his life felt that he was worshiping his Creator in a most accepted way by even the lowliest types of service in which he found himself of aid to his fellow men. A workman of distinction and varied acceptance in his chosen

profession, he also possessed a high-mindedness of soul and fineness of character—attributes not as common as they ought to be among men of the present time. Even in the day of his departure, his was the vision splendid of better service and greater usefulness in the years to come—an indomitable spirit of optimism, his home and work holding the center of his affections and dreams.

Mr. Tays was elected a Member of the American Society of Civil Engineers on April 6, 1898.